XXXIII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

AT A REGULAR MEETING OF THE SOCIETY, FEB. 21ST, 1872.

METHODS OF REDUCING THE COST OF RAILROAD CONSTRUCTION.

A paper read by Maj. ALFRED F. SEARS, Civil Engineer, Member of the Society.

The true, and, indeed, the only problem in American engineering is to ascertain the *minimum cost of properly attaining an object.* Doubtless we shall agree as to the main proposition, while there may be important differences concerning what is "proper" in cutting down expenditure.

It must be a matter of interest to us to reduce the cost of railroad construction to its lowest figure, so as to more rapidly bring this mode of transportation into its ultimate true relation to the common highway. Civilized nations will continue to use the earth road for horses and steam, in the transaction of neighborhood business; but the iron rail will be extended as rapidly as men can command the means of laying it down, so that an important duty for the civil engineer is to develop methods by which this great instrument of modern progress shall become more and more accessible to communities.

Most men hate to associate their names with a cheap thing; yet, I apprehend, the true ambition is to build the cheapest work.

There exists no physical difficulty worthy to be overcome for the purposes

of civilization, that cannot and has not been conquered by man. Nor is this success always the mark of genius. When men work for Governments with unlimited means at command, or for wealthy corporations, failure must be the exceptional fact.

It is natural enough to wish to build great works—works to be monuments to our fame. But sometimes we have need to guard ourselves lest, in this ambition, the problem which excites our chief attention be rather, *what is the greatest sum to be commanded for our purpose*, than the true one, *what is the least amount for which the work can be done?*

The books are written and records of experience given to show, not how to do a work, but how to do it in the best, that is, the cheapest way.

Already Mr. Fairlie has done much to facilitate the use of the hill grades and sharp turns of the common road by locomotive engines; and, more than this, has directed the public as well as the professional mind to a general revision of the systems of railroad building.

I venture to submit a few suggestions in the same direction, nor have I presented in this paper any plan or modification of prevailing customs to which I am not committed in practice.

I appreciate the danger of being misunderstood in the use of this word "cheapest." I am no advocate for the *slouchy* way of doing work, which is really an extravagant method. But I am forced to recognize the fact that railroads are in general demand, while their cost, as at present constructed, places them beyond the reach of poor and prudent communities. Vanderbilt's system is good for Vanderbilt's traffic; but the necessities of that system are exceptional. A road which is to be occupied immediately by a crowded traffic, cannot be built entirely in the manner prescribed in this paper, though its principles should assist them; but the thousands of miles of agricultural and local roads to be built may be made subjects of the savings I have here indicated, in their fullest extent.

Width of Road-bed.—The most important change that I have to propose concerns the width of the road-beds for the common gauge of $4\frac{7}{10}$ feet. It is the general custom to specify a uniform width of road-bed on embankments throughout the whole length of a line of road, and another uniform width for excavations. Occasionally the width at sub-grade is the same throughout both cut and fill.

I have abandoned this system as incorrect in principle and unnecessarily expensive.

On Banks.—The first element to consider in determining the width of a bank after getting sufficient for the length of a sleeper, is its height. We require only earth enough outside the tie to protect our road from the effects of rain-wash. Generally we find banks of a single foot in height, bearing the same width as banks with twenty feet of slope. Yet a slope one foot long is not exposed to one-twentieth part of the liabilities of a slope of twenty feet.

Suppose, to simplify the question, that no water is absorbed by the material, then the first foot * of slope below the road-bed is washed by just the amount of water that falls upon its surface ; the second foot by what has fallen upon its own area, and what has flowed over it from the foot above ; so that if a represent the amount of liability to which the first foot is exposed, $2a$ will represent the liability of the second foot, $3a$ of the third, and $20a$ of the twentieth foot ; while the total exposure of the slope is equal to the sum of the series ; a slope 20 feet long is therefore, under the hypothesis, liable to 210 times the damage that may accrue to a slope 1 foot long.

The books generally tell us that the common earths stand at a permanent slope of say $1\frac{1}{2}$ horizontal to 1 vertical, and further, that when a vertical bank first assumes a slope, it is curved—the lower portions being horizontal, while the top of the slope remains vertical ; but that eventually a uniform slope is formed, which becomes the permanent slope of the material. I confess that in twenty-five years of uninterrupted professional observation, I have never yet found the material which takes on a uniform slope, or anything approaching it, without the aid of a spade. On the contrary, a walk over any section of an American railroad will exhibit as many illustrations as there are considerable banks, of the principle that the natural slopes of all materials are curves. I suspect we have come, or rather been brought, to accept the commonly received notions on the subject by the fallacious system of averages; a mischievous instrument for any use except in the hands of the skilful, cautious, and conscientious.

Now, without entering on the useless trifling of ascertaining the curves of slopes, we may, by an observation of the actual fact, be assisted to very considerably economize work.

In banks a foot or two in height, we shall learn that a nominal slope for the purposes of the estimate is all that a permanent road-bed requires.

The embankments of what are known as surface roads are thrown up from side ditches, and are therefore amply protected from damage by water ; I build such banks 10 feet wide on top, when ballast is required, and specify a

* I use "foot" as a convenient unit.

slope of 1 to 1 as a definite limit to measurement. The ordinary custom in this region is to construct such banks 14 feet wide with slopes of 1½ to 1.

It is true that if a tie is 8 feet long, this width of bank will not allow much latitude for corrosion ; but the repair of such waste is always more cheaply made by a company in operation than before their line is opened for work; a proposition to be discussed hereafter.

Where my banks attain a height of more than 3 feet, I increase their width to 11 feet ; the additional foot being placed on one side of the centre line to facilitate the movements of the track-layers. I have found a berm of 1½ feet sufficient for all the movements required in track-laying. At the same time, I reduce somewhat the horizontal angle of the slope as the height of the bank increases : thus, a bank 3 feet high, is sloped 1 to 1 ; if higher, all between 3 and 12 slopes 1¼ to 1, and below 12, 1½ to 1.

In some materials, as in clay, I should use a slope as flat as 1½ to 1 for the light banks ; but in the ordinary earths I think it needless. It receives more water than a steeper slope of the same height, and does not permit as rapid escape of the flood.

In practice, the assistant on whom devolves the duty of setting slope stakes soon acquires dexterity in his calculations. Thus, for a bank 3 feet high, his top width is 10 feet. For a height exceeding 3 feet and under 12 feet, he deducts 3 feet, and proceeds as if the top width were 16 feet, with slopes of 1¼ to 1. Where the altitude of the bank is greater than 12 feet, he deducts 12 feet, works from a top width of 38½ feet, and sets his stakes for a slope of 1½ to 1.

Some assistants cross-section the ground, and make up the slope distances in the office.

The calculation of solids will not be found difficult, though involving somewhat more labor than in the case of simple slopes.

In Cuts.—I pass now to consider the methods of dealing with the *excavations* of a railroad.

The common system is to establish a uniform width of road-bed and interior side ditches, without reference to the depth or position of the cut. Several circumstances ought to modify these features.

A cut 1 foot deep cannot demand side ditches to carry the same amount of rain-fall that a cut 2 feet deep will require. Nor is there the same danger to be apprehended from the slip of the banks. Here are two reasons, then, for reducing the width of light cuts and also for modifying the slope.

Again, if an excavation is made through a flat plain, the two side ditches may be of equal dimensions; but if on a side hill, two circumstances affect this condition. The ditch under the upper side must drain a longer slope than the other, and is also liable to demand for draining the side hill above, when a berm ditch fails or a berm bank breaks; it should therefore be the more capacious ditch; yet even in such cases the side ditches are usually of equal dimensions.

In a cut 3 feet deep through plain, I make no excavations below sub-grade for ditches. If there be no ballast except such as is thrown between the cross-ties, there will result side drains 6 inches deep, which are sufficient for the purpose.

But when a steep side-hill cut is to be drained, a cut, with say 20 feet of slope on one side and about 2 feet on the other, I have sunk a ditch 6 inches below sub-grade with a top width of 2 feet under the long slope and provided none whatever along the lesser slope, except as in the case of a shallow cut through a plain, namely, such as is made by the ballast.

The capacity of the side ditch is further dependent on the grade assisting to drain the cut.

I apprehend that more width is usually given to road-beds in excavations than they require. In this region it is 18 feet, with side-slopes of 1½ to 1 or 1 to 1.

I am working considerably below this figure. Thus in gravel excavations of three feet or less I construct with a bottom width of 10 feet only and side slopes of ½ to 1. This will furnish a light gutter along each side of the track, and is all we require to keep it clear of water.

For a cut of more than 3 feet and less than 6 feet, I make a bottom width of 12 feet and slope of 1½ to 1. For a cut of more than 6 feet and less than 12 feet, 13 feet bottom width, and slope ½ to 1.

In this arrangement I have not designed finishing the cuts on compound slopes as in the case of embankments; my reasons for this course lead to a discussion of the question previously referred to, concerning the comparative cost of building a perfect work in the first instance and leaving it to be perfected by the company with a road in full operation.

It is a pet notion with the newspapers, and perhaps also with some engineers, that permanence in the first construction is true economy. It may be fairly doubted whether such is the fact, even when companies are possessed of what is called "ample means."

The entire excavation, so far as done, could be taken out for a less sum

per yard if the contractors could escape the very expensive operation of "trimming up" as usually executed by them. They are required to dress the slopes neatly; and then, to make sure of their standing, the engineer specifies a slope so flat as to insure its permanence beyond the possibility of accident. Now the ordinary earths will stand through a year, until the first spring, on a slope of ½ to 1. The general breaking up of things in the spring will bring down some of these slopes, creating a mass to be removed from the ditches, while many will remain standing permanently; such banks are clear gain.

At present prices, a gravel train will remove the detritus of the slopes for about 15 cents a yard; a work which could not have been done in the original instance by the medium of a contractor for less than twice that sum. It is done by the earnings of the road only, as it is demanded, instead of forming a dead capital for which the road is suffering annual damage in the shape of interest to be added to the shrinkage of its bonds.

I think engineers will eventually come to recognize that every dollar possible must be saved in the original construction of a work. Every unit of work on a road, which can be done after it is in operation, and the present neglect of which does not prevent the safe and economical movement of trains, should be so deferred. And this is simply a measure of proper economy, independent of the means of a company.

Allow me to illustrate by reference to the work I have in charge at this time. My road is 80 miles long. At 30 cents per yard, I find it will cost about $42,000 more to take out its excavations on a slope of 1 to 1 than if they were permitted to remain at ½ to 1, there being 141,000 yards more to be removed. Now, the cuts being shallow, the corrosion of the slopes will not interfere with the track; the ditches will occasionally be obstructed, but proper watchfulness and the exercise of fair judgment will generally prevent this inconvenience.

This amount of $42,000 will doubtless be increased to $46,000 by the shrinkage in bonds, a capital the annual value of which is $3,220. If then, the Company save the original capital, they will have this annual amount of $3,220 to apply to removing such material as falls from the slopes; at a cost of 15 cents per yard, they may thus take care of more than 21,000 yards per year. I believe this annual labor will keep pace with the corrosion of the slopes till the "trimmings up" is complete.

But the measure of economy is not yet full. In seven years, the slopes, if need be, will all have been reduced to the standard of 1 to 1, at a cost of $22,540, whereas, if taken out in the first instance, the actual outlay will be 20

years' payment of $3,220 interest, and then the principal, $46,000, or a total sum of $110,400. I have omitted mention of interest on the annual payments as being an element common to both cases.

Will you permit me one more illustration drawn from the same work? It concerns an important example occurring in less magnitude at several points of the line. We cross a ravine 140 feet deep, for a short distance in the centre, and reach grade on each side in a total distance of 700 feet. It will require 110,000 cubic yards of earth and a small arch culvert to build an embankment across the place.

Which will be the true economy, leaving out of view the present means of the Company? To construct the permanent work or trestle the valley, leaving it to be filled in hereafter?

The material may be procured sufficiently near the spot to build the bank for 40 cents per yard, or a total cost of $44,000. The culvert would probably cost $3,000, the shrinkage on bonds would require the expenditure of a capital of $53,000 to construct this work.

I can build a trestle of the timber of the country 550 feet long for $20 per foot, or $11,000, and put in the bank approaches for $2,000 more, or a total outlay of $13,000, requiring an expenditure of the Company's capital stock of $14,500. This trestle is good for ten or fifteen years. The comparative annual outlay will be as follows:

First.—On the temporary work:

7 per cent interest on the cost ($14,500)	$1,015
10 per cent. depreciation of trestle ($11,000)	1,100
Total	$2,115

Second.—On permanent work:

7 per cent. interest on $53,000	3,710

Annual difference in favor of the trestle system will be $1,595.

Suppose, then, we build the trestle and then apply the $1,595 annually to filling the ravine with permanent work. It will cost us about 12 cents per yard for the material, so that we can put in about 13,000 cubic yards per annum. In ten years we shall have filled the ravine and built the culvert instead of rebuilding the trestle, and we shall have done this out of the saving in interest money. When the job is done our capital is released to us for other work.

The total outlay in this case will be, for the first

Cost of trestle and approaches	$14,500
Ten years of interest and repairs	21,150
Total amount of annual outlays in filling ravine	15,950
Or, in all, a sum of	$51,600
Whereas, if the embankment be now built, it will cost 20 years' interest ($3,710)	74,200
And then the principal	53,000
Or a total amount of	$127,200

I estimate that the result of this system of construction saves to us in present outlay of original capital about $290,000, which, before it could be paid, would have drained the Company in twenty years of $696,000, or 3½ per cent. per annum of the capital stock, which, as the case now stands, we shall add to our income.

I have built no walls for cattle guards for several years. A common iron or wooden grating extending from fence to fence across the road, and 6 feet wide in the direction of its length, the grate bars being parallel thereto, is a sure protection against the invasion of cattle, and costs but little, compared with the ordinary methods. Though it has been in use by many engineers in parts of the country for twenty years, I still find men who build culvert masonry for cattle guards.

I invite the attention of the Society to one more feature of economy in the construction of works, which has been successfully tested in the vicinity of New York City. I refer to the light iron caissons in which were built the pivot piers of the drawbridges over the Hackensack and Passaic rivers on the line of the Newark and New York Railroad.

That road passing into the hands of the Central Railroad Company of New Jersey before I completed the work in them, I never had the privilege of watching their conduct to the end; but I have since learned that it was satisfactory.

The bottom of the Hackensack channel where it is crossed by the line is of sand, covered at high tide by 19 feet of water.

It was essential to avoid the difficulties which had pursued the Central Railroad Pier in the Bay below where the bottom was similar, and had been scoured by the action of currents so as to require constant watching and reinforcement with riprap. It was deemed expedient to build a permanent

work in masonry on a foundation of closely packed piles sawed off level with the bottom of the river.

The problem then was to build cheap masonry that would yet be permanent and provide a cheap method of depositing it. A coffer-dam would subject us to expense and to trouble in contending with water.

A wooden caisson of the requisite dimensions would contain 4,000 cubic feet of neatly squared lumber—must be securely built and caulked, and then would be liable to leak before the work was done, as the site was continually disturbed by the passage of steamers. I think the lowest offer we received for building such a work was $4,000.

After carefully weighing the matter I finally resolved to build a pier of concrete masonry in a caisson formed of No. 10 boiler plate or sheet iron.

The iron men and the contractors cautioned the Board of Directors against the folly of such an erection. They declared that a cylinder of iron but $\frac{1}{10}$ inch in thickness and 32 feet diameter could not be built 22 feet high; that if built, it could never be safely launched, or if launched, could not be towed to its position from the iron yard. They were sure that if it reached its berth the masonry would stave through the bottom of such an egg-shell. The Executive Committee hesitated.

I had recourse to the friendly counsel of Mr. Wm. H. Talcott, then President and Chief Engineer of the Morris Canal. His goodness and wisdom had never failed me. He sustained my views and the Directors were satisfied.

The caisson consisted of a cylinder of No. 10 iron, closed at the bottom and built with single rows of rivets to a height of 14 feet. The sheets necessary to make up the remaining 8 feet were punched and fitted, then numbered and taken down to be put in position as the vessel sank to its place on the level top of the piles; 8 feet above the bottom of the cylinder were two diametric rods at right angles to each other, to check the distorting effect of motion in the water; and under these rods light wooden struts extended to short straining timbers on the bottom, to resist the bulging that would be caused by the reaction of the water on the outside. This secured a level base for the masonry. From the ends of the tie rods on the outside of the caisson projected four eye-bolts.

I completed the wooden guard pier which accompanies the pivot-pier to receive the drawbridge when open, before launching the caisson, leaving one side open to receive it on arrival. When this vessel was launched, a very

simple operation, since it weighed but 9,000 lbs. it fell about three feet from the end of the wharf into, or rather upon, the water, and bobbing about like an empty band-box finally settled upright, sinking about eleven inches in depth. It was towed by a steamer to the guard pier, distant about four miles, floated into its berth, and a single sheave watchtackle made fast to each eye-bolt, connecting it with the surrounding wood-work of the pier. These lines were to regulate its sinking to its place, keeping it level and holding it quiet when steamers passed; they answered the purpose for which they were designed.

Again, the contractor protested he was sure the first yard of masonry would go through the bottom of such a flimsy affair; he was "a practical man and no theorist, and wasn't to be fooled with any high sounding talk about the pressure of fluids; he knew that water wouldn't hold up a stone anyhow."

Finally, the danger of laying the first course was shared by myself or an assistant remaining in the caisson, till the men became assured and ceased to require such a guaranty for their safety.

The disturbance of passing steamers caused some leaking, but a common kitchen hand pump worked by one man secured perfect immunity from water.

Each foot of masonry sank the caisson two feet, and when it rested in its place at high tide it contained ten feet of masonry and an empty chamber of twelve feet in height. We found it expedient at this stage to fit into it a light centering about the level of the high water mark, to protect our masonry from displacement when the caisson was disturbed. The whole job, both caisson and masonry, was the cheapest of the kind I have ever known. The subsequent testimony of the Contractors, Britton, Cole & Stewart, was that they had never built masonry below the water line so comfortably and cheaply, the men working dry-shod, and the work all in sight, with no expense for pumping.

My design, I need hardly say, was to build a caisson that would last only such length of time as should be necessary for the perfect induration of the masonry, which was laid as follows :

The bottom course, 2 feet thick, was a beton of cement, sand, and the trap rock of Bergen Hill, broken to pass through a three-inch ring.

The second course, also 2 feet thick, consisted of an outer ring of the beton 3 feet wide, the remainder of the course being rough rubble laid in good hydraulic cement mortar ; and this arrangement—courses of beton alternating

with concrete courses of beton and rubble—constituted the masonry of the pier, only the bridge seat being of cut dimension stone.

I may here properly state my reason for this arrangement of masonry, which was, to use up the most accessible and cheaply procured material of a neighboring quarry. Such stone as was too large for beton without considerable labor, being used in the rubble work.

When the caisson had fairly settled to its bed on the piles, the foundations were surrounded by riprap, as shown in the drawing, to prevent the scouring of the sand bottom.

I have not seen the pier in some months, but the last account I had of it was that the iron, having served its purpose, has disappeared in considerable portions, and reveals the body of the masonry intact, while the foundations are unchanged.

When the work passed into the hands of the New Jersey Central Railroad Company, Col. James Moore became the Engineer and built the Passaic pier in the same winter. He also used the same method with the abutments of the drawbridges, which were rectangular, and therefore not so well adapted to the employment of very light iron.

I should not, I think, have ventured on the experiment with any other than the cylindrical form, except as a cover to a sufficiently stiff frame of wood.

I presume that masonry for sub-aqueous positions in light iron caissons with wooden frames, may be built at less cost than by any other known system, though the general practice favors wooden coffer-dams. The character of masonry used in this pier has been often built in Europe on the Continent, and I believe to some extent also in the British Islands.

I think, however, the usual course has been to alternate a concrete course of stone and beton with a course of dimension stone—the beton forming only a backing to a cut ring.

So far as my experience extends, and it is considerable, I would not hesitate under any circumstances to use beton as outside work, if I could command a good article of cement at fair rates. If it can be laid without exposure to wash, it becomes harder and tougher than some of the building stone and brick.

The parapets on the bastion of Fort Clinch, Florida, were filled with beton of the same brand of cement that I used in the Hackensack River piers, "The Hudson River Company's;" after standing a year, orders were received for their removal to adapt the work to late improvements in Ordnance, and so

thoroughly were they indurated, that it became necessary to use drill and blast to accomplish the work. Such masonry preserved for a year in light iron caissons from the action of water in submarine works, will be found equal to all requirements.

While I should be glad, gentlemen, to believe that these suggestions may assist to reduce the cost of constructing railroads, I am forced to recognize a certain class of obstacles in the way of the Engineer, that he struggles against in vain, and must continue to so long as the present extravagant system continues, of letting works in the lump.

These obstacles result from the combined ignorance, conceit, and corruption in Boards of Direction having to deal with a certain class of public securities, such as State, county and municipal bonds, whereby all the machinery of political jobbery and cunning is set to work and the conscience of the Company is debauched before operations are fairly commenced.

A common practice is to set forward as president a simple-minded, credulous man, of fair reputation for integrity, who thus possesses the confidence of the community, and whose vanity and weakness of mind make him the tool of a smart bad man, the manipulator of the Executive Committee. The Engineer, placed thus between a knave and a fool, soon finds himself at the mercy of a single man, unscrupulous, tyrannical, and successful. He learns that his Board by their own action have been reduced to "one mind"—the conduct of which he is not to observe too closely or he must resign in half disgrace.

In any event the result on the cost of building his work is the same. The knave by consent of the Board lets the work to a confidential associate, a railroad contractor, at an outrageously high figure. The Engineer's plans of economy are overslaughed, and he is finally pushed aside for the opinions of a man, who, though ever so ignorant, being a contractor, is supposed to possess superior *practical* knowledge of railroad construction, and who is quietly putting the Company into his pocket.

There is probably not a road in the State of New York now under construction built in the lump, with town bonds, that is not being fraudulently depleted to the extent of from five thousand to twelve thousand dollars per mile before the superstructure is touched.

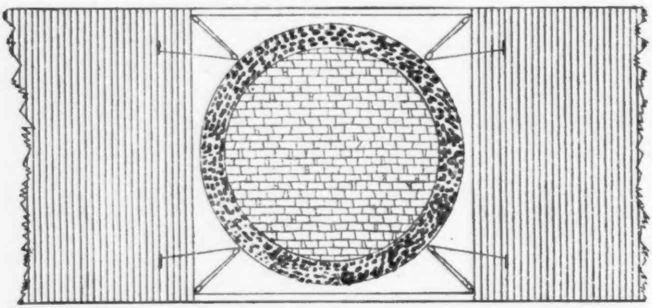
And yet, in all the revelations of fraud for which the day is conspicuous there appears no apprehension of this condition of things.

One protection against this outrage, is to be found in the old practice of

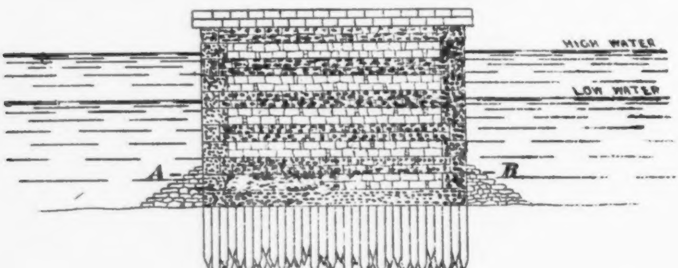
letting the work by the cubic yard, which is the only safe method and presents the only fair way of receiving the bids of contractors.

But when this corruption shows itself in a Board, the Engineer should expose and denounce it as soon as it appears, though at the risk of defeat and dismissal—a result that would not many times occur, since the public attention would be aroused and the Engineer would soon come to find himself occupying his true relation to the Company that employs him.

Drawing showing Position in Guard Pier.

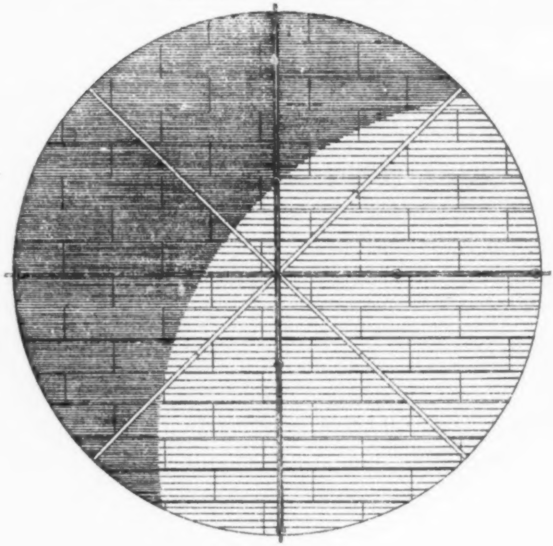


Section of Pivot Pier on line A B with Projection of Guard Pier at Floor.

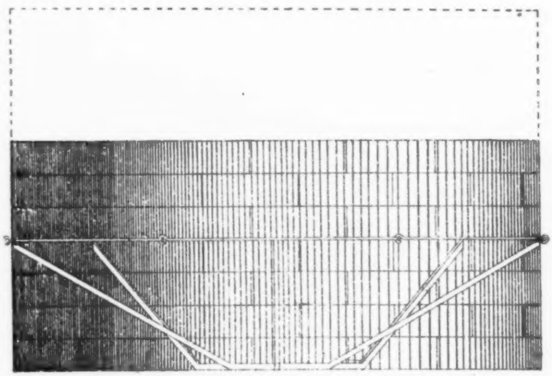


Vertical Section of Pivot Pier.

Iron Caisson of the Pivot Pier of the Hackensack Draw Bridge,
Newark and New York Railroad.



Plan and Projection.



Vertical Section and Projection.

XXXIV.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT A REGULAR MEETING OF THE SOCIETY, MARCH 6TH, 1872.

AN ANALYSIS OF THE COST AND DESCRIPTION OF THE METHODS OF MINING EMPLOYED IN THE MARQUETTE IRON REGION, LAKE SUPERIOR, MICHIGAN.*

A paper read by MAJOR T. B. BROOKS, C. E., Member of the Society.

The product of the Marquette Mines has nearly doubled during the last four years, and is now about 825,000 gross tons of ore, which will yield on the average 62½ per cent. of pig iron in the furnace; hence, during the last two years, fully one-fourth of all the pig produced in this country has been from Lake Superior ores. About 66 per cent. of the shipments are specular hematites, yielding 65 per cent. of iron; 17 per

* This paper, written in 1870, forms a part of the 2d article of the Chapter on Mining of the writer's unpublished Report on the Geology of the Marquette Iron Region. A portion of the 1st article relating to the geological structure of the ore deposits, and a discussion of the general mining problem presented, was read before the American Institute of Mining Engineers. That paper contained the skeleton of this.

cent. are magnetic ores, yielding 65 per cent. of iron; 13 per cent. are soft hematites, yielding 50 to 55 per cent. of iron, and 4 per cent. second class specular ores, yielding, say 57 per cent. of iron. One hundred and twenty-five furnace stacks use Lake Superior ores wholly or in part; of these, 89 employ coke or bituminous coal as fuel, 10 anthracite coal, and 26 charcoal. The charcoal furnaces are all small, seldom producing over 5,000 tons per annum. Eighty of the bituminous and coke stacks are in Ohio and Pennsylvania;—all the anthracite stacks are in Pennsylvania and New York, and over one-half of the charcoal furnaces are located in Michigan. The iron produced is soft and strong, answering equally well for mill or foundry use. It inclines to red-shortness without being decidedly red-short. It is too soft for rail-heads, but is unequalled for the base of the rail and for merchant bar, and is now being successfully used for Bessemer steel.

The geological structure of the iron-bearing rocks is such as to render quarrying in open cuts the favorite method of extracting the ore. The ore formation is made up of pure ore, associated with lentiform masses of a soft green schist and of jaspery or mixed ore. The overlying rock is quartzite, having a specific gravity of 2.67, and the foot wall rock a diorite or greenstone, having a specific gravity of 2.91. The merchantable specular and magnetic ore has a specific gravity of from 4.59 to 5.00, averaging 4.85, weighing therefore about 300 pounds to the cubic foot, or 3½ gross tons per cubic yard. Seven and a-half feet of solid ore and about 15 feet of loose ore, as it is sent to market, weigh one gross ton. The specific gravity of the "mixed ores" varied widely, depending on the amount of jasper, the average being about 3.75. The soft green schist associated with the ore has a specific gravity of 2.82.

The unit of measure and comparison in the following table is the gross ton of merchantable ore. The ore is the object of the miner's efforts, and the tons sold measure his business. The items of cost in all that follows express the expenditure per ton of ore mined, prepared for market, and loaded on the cars. In instituting a comparison between these figures and those obtained by the civil engineer, where the cubic yard of vacant space is the ordinary unit of work accomplished, it must be borne in mind that the labor incident to sledging up and sorting out the ore from the rock considerably enhances the cost of mining.

The wages of the men employed in and about the mines, in 1869 and 1870, were about as follows: Common labor was nominally $1.80 per day for most of the time, but by far the largest part of the mining work was done under contracts. Contractors made, clear of costs, from $60 to $77 per month as high and low averages; $70 is probably near the mean of the whole. It is not uncommon for a "pair," two or more men, to make $100 per month each, and again the earnings will fall so low as barely to pay board; but such are extreme cases.

Leaving out the staff of the mine and the contractors, the average wages of all others, mechanics, engineers, firemen, drivers, but mostly common laborers, averaged for the period in question about $2.12 per man per day. Mechanics received from $2.50 to $4.00. The nationality at three mines, which employed an aggregate of over 600 men, was as follows, expressed in percentages:

Irish	31
English (Cornishmen)	27
Swedes	18
Canadians (French)	5
Americans	5
Germans	4
Norwegians, Danes, and Scotch	10
	100

The relative proportion of the Irish element is decreasing; a few years since nearly all the men employed at some mines being of this nationality. The percentage of Cornishmen is increasing, owing largely to a want of work in the copper region. These men are skilled miners and do a large part of the sinking and drifting. Swedes are rapidly gaining in numbers, many of them having been miners in their own country.

The $2.64 given in the following table as the approximate total cost of the hard ores, delivered on the cars at the mine, was obtained by dividing the total expenditure at the mine for the year by the total number of tons of ore produced. It does not include interest or capital, nor royalty, or depreciation of the mine.

TABLE *showing the Approximate Cost of Mining the Specular and Magnetic Ores of Lake Superior (to accompany Major T. Brooks' Paper on the subject).*

General heads under which cost of mining is classified.		Elements of cost, not including royalty or depreciation.	APPROXIMATE COST OF EACH ITEM. In per cent. of the whole. Items.	Totals.	Based on a total cost of $2.64 per ton. Items.	Totals.	Amounts. Labor.	Supplies.
I. Dead work (preparation..........		1. Explorations	.006	.281	.015	.742	Eighty per cent. .620	Twenty per cent. .122
		2. Sinking Shafts.......	.015		.040			
		3. Drifts and Tunnels..	.061		.160			
		4. Roads	.006		.017			
		5. Stripping earth and rock	.132		.350			
		6. Miscellaneous work and minor improvements*	.061		.160			
II. Mining proper (labor).	Drilling.	1. Ledge holes (in stope)..............	.042	.398	.110	1.050	1.050	....
		2. Block holes (in fragments)............	.049		.130			
	Other work.	3 Sledging, sorting and loading........	.133		.350			
		4 Handling rock.......	.095		.250			
		5. Miscellaneous work.	.079		.210			
III. Mining materials and implements ("mine costs").	Explosives.	1. Powder and fuse....	.036	.119	095	.313	.103	.210
		2. Nitro-glycerine.....	...†		... †			
	Tools.	3. Steel (drills)........	.007		.018			
		4. Tools other than drills..............	.016		.043			
	Repairs.	5. Blacksmiths' supplies...............	.018		.047			
		6. Blacksmiths' labor...	.042		.110			
IV. Handling ore from miners' hands to cars, and pumping.	By horses.	1. Teaming, labor of drivers and stablemen................	.057	.156	.150	.413	.272	.141
		2. Forage.....	.042		.110			
		3. Carts, sleds, harness, &c............	.002		.006			
	By men.	4. Loading ore from stock pile..........	.013		.035			
	By steam.	5. Labor, supplies and repairs............	.042		.112			
V. Management and general expenses.		1. Salaries and office expenses......... 2. Tax of all kinds...	.046	.016	.122	.122	.062	.060
			100	100	2.64	$2.64	$2.107	$0.533

* Does not include exceptional permanent improvements.

† No reliable figures obtained

In the detailed description of methods which follow, the items will be taken up in the order of the table, omitting such as are unimportant, or do not possess the kind of interest that warrant their introduction here. The whole subject will be considered more fully in my report on the Lake Superior Iron Region.

1. *Dead Work.*—This general head embraces all the work and costs incident to getting ready to mine the ore, and is subdivided into—1. Explorations (embracing only such searches for ore as are in progress from year to year about the mine). 2. Sinking Shafts. 3. Drifts and Tunnels. 4. Roads for Wagons. 5. Stripping Earth and Rock, or uncovering the ore. 6. Miscellaneous Work and Minor Improvements. Only the 2d, 3d, and 5th items will be considered, the character of the others being sufficiently indicated in the table. The entire expenditure for dead work is 74 cents per ton of ore produced, which equals 28 per cent. of the whole cost.

2. *Sinking Shafts.*—This work, which forms so large an item of cost in some underground mines, varies in the Marquette Region, so far as I have ascertained, from 1½ to 5½ cents per ton of ore. Our open and comparatively shallow workings do not call for many shafts or winzes; the deepest shaft in the Region is now (1870) not over 200 feet. The prices for this work range from a mean $22.50 to $31.50 per foot in depth, depending on the hardness of the ground. In some mines, extreme prices range from $15.00 to $40.00; or even more, if the shaft be very wet. Miners are often permitted to select the size most advantageous to themselves, which may be four feet by six; but eight by twelve feet is more common. The material is generally hoisted with the ordinary hand windlass, but sometimes with a horse-whip or whim. The miner has to deliver the stuff at the mouth of the shaft. From 10 to 15 per cent. of the price received by the miner for sinking, has to be expended in costs; *i. e.*, powder, fuse, candles, steel, tools, etc. No charge is made against him for smith's work. Sometimes the contract is let at so much per foot of shaft and so much per ton of ore, which gives the miner an interest in separating ore from rock.

3. *Drifts and Tunnels.*—This element of cost varied more widely than any other, and might have been divided into two: (1) Drifts designed to open ground for stoping, and (2) Tunnels or adits for drainage and

transportation of ore, the latter being of the nature of a permanent improvement. But on the principle that permanent improvement accounts are often permanent disappointment accounts and to be avoided, and considering the fact that this work is actually going on year by year and must do so as long as the mine is worked, it does not seem wise to separate it from the actual cost of getting ore. Ordinary 4 × 7 drifts cost, in hard ore, from an average of $22.50 to $24.50 per foot, the miners delivering the material behind them and paying their own costs as in the case of shafts.

Tunnels large enough to admit railroad cars and small locomotives, cost from $30.00 to $50.00 per foot. The Washington Tunnel, now over 1,100 feet long and timbered considerable part of the way, cost an average of about $40.00, not including rails. The timbered portion is twelve feet wide at the bottom, ten feet at the top, and ten feet high in the clear. No machinery has yet been brought to bear on either sinking shafts or drifting, the labor is more than one-half expended in drilling holes for blasting. The subject of drilling is fully considered under its proper head.

5. *Stripping Earth and Rock*, or uncovering the ore. This constitutes on the average nearly one-half of the dead-work, and is one of the largest single items in the whole cost of mining. So far as my inquiries extended I found it to vary from 20 to 52 cents per ton of ore. This cost is necessarily increasing at all of the mines worked as open cuts. It is simple rock and earthwork, the material being removed on wagons, carts, or sleds, drawn by horses. The advantages of light railroads and small locomotives do not seem to have commended themselves for this work. There would of course be considerable danger of destroying tracks from blasting, and it often happens that not much work has to be done in one place; still there is no doubt but that a large saving would be effected by substituting steam for horses in this work, as will be more fully considered hereafter.

The aggregate amount of material which has been handled in stripping is very great. Thirty, and even forty feet of earth have been removed, and nearly as great a depth of rock; but this is the experience in open workings everywhere. I have seen twenty-one feet of earth and soft, shaly rock stripped from a nearly horizontal bed of 44 per cent. Clinton ore in Western New York, which did not average over thirty

inches thick. In South Eastern Kentucky I found the rule among the miners of sub-carboniferous ores to be, that it would pay to remove a foot of earth for the sake of an inch of ore, which does not differ widely from the Western New York practice. In both of these instances the stripping was nearly the entire cost of mining, and labor was much lower than in the Marquette Region. The usual contract price for removing ordinary earth (sand, clay, and boulders mixed together) is fifty cents per cubic yard, the digging costing about one-half and the handling one-half. Hauls vary from 100 to 800 feet. The highest price paid for excavating any considerable quantity of rock in open cuts, which has come to my notice, was $3.00 per cubic yard, equal to $24.00 per fathom, or about $1.00 per ton. This was a very hard jasper rock containing but little ore. Large quantities of rock have been excavated and hauled over 500 feet at the Lake Superior Mine for $2.50 per yard. The soft greenish schist, so common at all the mines, can be moved for from $1.00 to $1.40 per yard, including hauling. When a good face can be obtained on the overlying quartzite, which is likely to constitute the greater part of the rock to be moved in future, it should be broken down and loaded on wagons for $1.50 per cubic yard.

The amount of money which it will pay to expend in stripping of course depends chiefly on the quantity of ore uncovered. If we assume fifty cents to be the maximum expenditure per ton of ore for this work (this amount has been exceeded), the problem of what thickness of rock may be stripped admits of an easy theoretical solution. One cubic yard of solid ore, allowing for wastage on account of associated rock, may be considered to yield three tons of merchantable ore, which, at the allowance above assumed, would give us $1.50 to be expended per square yard in stripping a bed of ore only one yard thick. Hence in this case it would pay to remove nine feet in thickness of earth or about three feet in thickness of rock. But suppose we have a bed of ore twenty-four feet in vertical thickness, which is a more common case, what amount of rock or earth would it pay to remove under the assumed limit of expenditure? Twenty-four feet of ore gives twenty-four tons per square yard of surface, which, at fifty cents per ton, gives $12.00 available for stripping per square yard. This sum would remove twenty-four feet thickness of solid rock; or a foot in thickness of

rock may be stripped for every foot in thickness of ore uncovered, at a cost of fifty cents per ton of ore. The same expenditure will remove three times this thickness of earth.

An important and often neglected question connected with this subject, is, where to deposit waste, that it may be out of the way of future mining operations. Some material has been already handled twice in the Marquette Region, and I know of a mine in Southern New York, where the same earth has been three times handled before it was finally put out of the way. In a new region, like Marquette, where comparatively little thorough exploring has been done, it is often difficult to decide where waste piles will be out of the way for all future time. If a drill hole were put down for fifty feet in rock and no ore found, it would be safe to say that if ore existed under that spot, it would have to be mined under ground; hence, that so far as future stripping was concerned, a waste pile placed there would be out of the way. A very common practice in under-ground work, in some mining regions, is to fill up the worked out places with waste, and this can undoubtedly be done to advantage in some instances in open works, although it has not as yet been practised in the Marquette Region. The trouble is, to find out when a pit is exhausted—it is so common to break through a thin layer of rock and find a bed of workable ore under it. But there are parts of most mines where the foot wall has unquestionably been reached, and if any doubt exists, a few deep drill holes will settle the point. When this is the case, and the foot wall has a sufficiently gentle slope to permit of its holding materials deposited on it, it will, I think, be often found advantageous to use it to support a waste pile.

For the sake of illustration, take the New York and Cleveland Mine workings, which are adjacent. In this instance the slope of the foot wall is so steep that it would probably require a rude step to be cut on which to rest a rough retaining wall, built of blocks of quartzite swung across from the hanging wall by means of a derrick. The triangular space thus formed would hold all the waste rock for a long time to come, and would afford a minimum haul. It might not answer to deposit earth in such positions, as heavy rains would be likely to wash it into the pits. The dip of the foot wall in this, as well as in most cases, will I think become flatter in depth, so that a better opportunity will be afforded for a second similar waste receptacle at greater depth,

if one should be required. This plan would also have the advantage, when under-ground work is begun, as it soon will be, of affording a good support to the roof of the mine.

II. *Mining Proper, or Breaking Ore.*—This general head embraces all the labor incident to blasting the materials down from the solid ledge and breaking it up into fragments that may be easily handled, and the separation of the ore from the rock by hand. The average cost under this head is $1.05 per ton of ore produced, which equals forty per cent. of the whole. The character of this work will be sufficiently well understood from the table and the following explanation.

1. *Ledge or Stope Holes.*—The drilling or rock-boring is now entirely done by hand. The steel used for drills is 1¼ inch octagon with a bit 1½ inches, making a hole nearly two inches in diameter. Drills vary in length up to 24 feet. English steel is used at some mines, but a majority use American steel, and the most experienced men who have used both inform me that the drill steel made by Hussey & Wells and Parke Bros., Pittsburgh, is about as good as the best imported steel, and much better than the average. The drill is turned by one man sitting and struck by two standing, with eight pound hammers, at the rate of thirty-six blows per minute each. In this way from nine to eleven feet of hole are sunk per day, the men working usually on contract. The price of stope holes ranges from 60 to 80 cents per foot in depth, the mean being not far from 75 cents; no mine costs have to be paid out of this price. When there is a large proportion of block holes, which admit of the use of smaller steel, the whole drilling of a pit is often let at from 60 to 65 cents. Very deep holes, say from fifteen to twenty-two feet, are sometimes sunk with two-inch bits, which about doubles the cost. In these cases, two men sometimes turn the drill and three strike.

The cost of drilling ledge holes per ton of ore, varies from a mere trifle in the case where one twenty-two foot hole throws down 4,000 tons, as has been done, to a very large item on low stopes with perhaps tight, hard ground. From three (3) cents to 25 cents per ton may be regarded as extreme averages, although 35 and even 48 cents have been reached, for short periods, under very unfavorable circumstances. The price given in the table (11 cents) approximates to the

average for hard ores; this number, divided into 75 cents, the average cost of drilling per foot, gives say 7, which should represent the number of tons of ore broken per foot of hole drilled. The data obtained directly under this head confirm this amount, which is equivalent to about two cubic yards per foot of stope hole.

The depth of stope holes varies from two to twenty-two feet, the short ones being employed in "taking up bottom," that is, in squaring the stope so as to give the best chance for the deep holes. The average of 1,500 stope holes of all kinds in one part of the Washington Mine was four feet nine inches, but the stopes which furnished this result were below average height. It is believed that nine or ten feet would be nearer the average for deep holes, and say 3½ feet for the short ones.

2. *Block Holes.*—The masses of rock and ore loosened by the heavy blasts already described, are often so large that they have in turn to be broken with explosives, which operation is termed block-holing. The amount of this work varies from almost nothing in some pits and some mines, to four-fifths of all the drilling required in others, the maximum being reached on high stopes of hard, tough ore. Over two hundred block-holes have been required to one stope hole in the Cleveland Mine, one hole being required to every two to four tons of ore. Block-holes sometimes produce fragments so large as to require block-holing in turn, before they are made small enough to be mastered by the sledge. Block-holes vary in depth from eight to twenty-four inches, the mean ranging near one foot. With nitro-glycerine the holes need not be so deep as for powder. One inch octagon steel is used in this work, making a hole nearly 1¼ inches in diameter. The drilling is performed as in the case of deep holes, but usually only one man strikes.

In the same ground, the same drill gang will sink more than twice the number of feet of block hole in a day with small steel, than of stope hole with large steel,—ranging from twenty-four to twenty-seven feet. In open mines of strictly hard ore this work costs more than stope holes, and is set down in the table at 13 cents per ton. This amount, added to the 11 cents given as the cost of stope holes per ton, equals 24 cents for the total cost of the labor of drilling required under breaking ore:—this

would equal about 70 cents per cubic yard, which would pay for one foot of two-inch drill hole. But this is by no means the whole; the work of sinking and drifting, which is set down as aggregating 20 cents, is more than half drilling; and a part of the cost of rock stripping is also for this work. I estimate that 40 cents per ton of ore is not far from the actual price paid for this kind of labor in the hard ore mines, equal to fifteen per cent. of the whole cost. On this estimate, not less than $300,000 were paid out for drilling in 1870. This work, from the favorable circumstances under which much of it is done in open excavations, no scaffolding being required, is by far the most mechanical labor performed about the mine. While the absolute cost of this item of drilling is very large, and can undoubtedly be reduced by the use of the power-drill, it is, as compared with some other mines and regions, small. Our open cuts or quarries afford far better facilities for blasting than underground mines. In one Southern New York mine the drilling cost, in 1870, $1.25 per ton of ore, or forty per cent. of the whole cost of mining; in a large magnetic mine in New Jersey, it cost from 60 to 80 cents per ton of ore. In the Presberg mines, Sweden, when the ore cost, in 1870, $2.20 currency per ton, the drilling was 40 cents per ton, equal to twenty-three per cent. of the whole cost, being considerably more than ours absolutely and relatively. When we consider that the average wages in Sweden is not far from 50 cents per day, or say one-fifth of what is paid Lake Superior miners, it would seem as if Sweden would be a good field for a power-drill.

3. *Sledging, Sorting, and Loading.*—In considering this item, it must be borne in mind that the ore and rock have not only to be broken so that they can be removed, but so fine as to be easily separated, and so that the pieces can be fed into a Blake crusher. This work requires more muscle and as much skill and care as any other done at the mine. Twenty-three pound sledges are employed, and the difference in results, between the experienced miner who strikes the lump of ore the right blow in the right place, with this immense hammer, and the tyro, is very great. Contracts for sledging and loading, sometimes including a little block-holing and short tramming, have been let at prices varying from 20 to 50 cents per ton. The loading usually costs not to exceed 10 or 12 cents, the balance being chiefly sledging. There is a wide difference in the texture of ore, some kinds requiring five times as much sledging

as others. On the whole, Marquette ores break with much greater difficulty than those of the Eastern magnetic mines. With poorer ground worked and the market more in favor of buyers (which makes them more exacting on quality), the cost of this element will be increased.

III. *Mining Materials and Implements, embracing "Mine Costs."*—This general head is sub-divided in the table into Explosives, Tools, and Repairs, which are in turn itemized, as will appear below. The expense incurred under this head is 31⅓ cents per ton of ore produced, equal to about twelve per cent. of the whole cost.

1. 2. *Explosives.*—Powder and fuse and nitro-glycerine. The present (1870) is an unfortunate time to collect statistics regarding the cost of explosives, for the reason that nitro-glycerine is on trial, and most of the mines employ both it and powder in the same pits, making it difficult to separate the results. The place of the new explosive cannot be said to be fixed in our mines. It is more powerful than powder, bulk for bulk, or weight for weight; can be used in wet as well or better than in dry ground, which is very important in some places; it has so far proved no more dangerous than powder, and its fumes have not been found objectionable. As has been stated, the fragments resulting from its use are usually smaller, hence require less sledging, and it being more powerful than powder, less drilling is needed. I have not sufficient data to institute a comparison between it and powder for general use: for very wet or very tight ground, and for much of the sinking and drifting, it answers better than powder. Whether it is suited to breaking the great masses from the solid ledge remains to be seen. Certainly it cannot be used to fill the cracks produced by shaking, when heavy sand blasts are required; and it is doubtful whether drill holes large enough to contain the requisite amount of the blasting oil can be profitably employed. It certainly costs *more* per ton of ore mined than powder, but how far this greater cost is balanced by other advantages remains to be seen. It is significant that in 1870, being the next year after its introduction, over $70,000 were sold in the Marquette Region at $1.50 per pound. The Painsville Ohio Co. have erected (1871) a factory near Marquette. No other explosives than powder and nitro-glycerine have been used.

The figures given in the table, and what follows, refer exclusively to powder, the nitro-glycerine element having been eliminated as far as was

possible. Fuse costs about ½ a cent per ton, which leaves 9 cents per ton for powder, which according to the data obtained varied from 7 to 10 cents. The price of powder ranged from $3.75 to $4.50 per keg of 25 pounds. Therefore an average of 45 tons of ore should have been broken with one keg of powder, or about ½ pound of powder to one ton of ore. This, it must be remembered, does not express the actual work of the powder, on account of the amount of rock moved in addition to the ore—in one instance 23,000 weighed tons of material required 320 kegs of powder, or 72 tons per keg. In another instance 31 kegs threw down 3,500 tons (approximate) of quartzite, or 113 tons per keg. One mine, which produced over 100,000 tons of ore in 1869, consumed for all purposes one keg of powder to every 43 tons of ore produced. The waste material in this case did not amount to over 20 per cent., hence about 52 tons, or, say, 18 cubic yards of material, were moved per keg of powder. The consumption of explosives per ton of ore must increase as the mines grow deeper, either by the greater amount required to remove the rock covering, or by the less favorable opportunity afforded for blasting, if the ore be won underground.

In one group of New Jersey mines, the powder and fuse in 1870 cost 18 cents per ton; in another mine in Southern New York, 14½ cents; in Sweden, at the Presberg mines, 15 cents. All of which figures considerably exceed those reached in Marquette, which is an additional proof of the economy of working iron mines as open quarries as long as possible.

3. *Steel.*—The use of steel drills has already been described, and reference made to the brands in use. My data, which are far from complete, under this head, indicate that the cost of steel per ton of ore ranges from $\frac{3}{4}$ to $3\frac{2}{10}$ cents, averaging perhaps $1\frac{8}{10}$ cents; the price of steel being 20 cents per pound. This would give about 11 tons of ore, or about 3 cubic yards per pound of steel consumed, which is less than the data obtained direct on this point seemed to indicate.

It is the practice at some mines to charge the ore contractors 2 per cent. on their contracts for wear of steel, which agrees nearly with the above. At other mines the steel is weighed at the end of each month, and the contractor charged with the shortage, whatever it is.

IV.—*Handling Ore from Miners' hands to Cars, and Pumping.*—The pumping, which has heretofore been a small item in the Marquette Region, could not well be separated from the hoisting of ore, as the same machinery does both. This item in the case of some New Jersey magnetic mines costs 75 cents per ton of ore: at the Presberg mines, Sweden, it costs but 7 cents. The entire cost under this head, including hoisting and pumping, is 41 cents per ton of ore produced, which equals 15½ per cent. of the whole. This work is done in part by horses, part by men, and part by steam.

1, 2, 3. *The Work of Horses in handling Ore.*—The team work employed at the Marquette mines, apart from the stripping, amounts, according to my inquiries, which have been quite full on this point, to ten per cent. of the whole cost of mining, or say 27 cents per ton, the drivers' wages being the largest item. This cost is obtained by dividing the total expenditure for teaming by the total number of tons of ore produced. If it was figured only on the ore actually handled by horses, it would be much greater. If to this were added the cost of the team-work employed in stripping, the total would not be less than 30 cents per ton of ore, or say $250,000 on the product of 1870, a sum sufficient in itself to supply all the mines in the region with all the additional steam hoisting and pumping machinery and small locomotives required to do the work now done by horses, and at a very much less yearly cost. We may verify this almost incredible estimate in another way. The total number of horses employed at all the mines in 1870, including hired teams, averaged about 364, or 30 to each mine, varying from 9 to 74. The best data I could get indicate that to run a lot of horses for one year, including wages of drivers, stable-men, smiths' work, forage, repairs of vehicles and depreciation, in the years 1869 and 1870, cost an average of $650 per horse. The wages of hired teams, including drivers, for the same period, was $6 per day. At this rate, 364 horses would have cost nearly $240,000, a sum sufficiently near the other to confirm the general truth of the estimate.

These figures surely justify the prediction that if there ever comes a period when our mines do not pay, it may be due largely to horses. In this age of steam, has a business any just right to prosper which employs horses to do work that can be more cheaply done by machinery? The average number of tons of ore handled per horse employed in and

about the mines for all work in 1870 was 2,350, ranging from 1,150 to 5,300 tons. In considering these facts it must be borne in mind that the mines n question are not by any means without steam power. Twelve engines, varying in power from say 10 to 50 horse, were at work. To prove that this item of cost is unusually large in the Marquette Region, I will give a few facts regarding the employment of live stock at mines, which have come under my notice elsewhere. While the cases cited do not present all circumstances like the Marquette mines, they are sufficiently near to afford interesting comparisons.

The Cornwall Ore Bank Co., Penn., shipped from their one immense deposit, in 1870, over 174,000 tons, employing no horses in the work. The ore was all handled by one locomotive, the cars being loaded by wheel-barrows. No pumping is required in this mine, and the facilities for reaching the ore with cars are unusually good. The ore is quite soft, so that the blasting does not endanger the tracks.

The Iron Mountain Mine, Missouri, shipped, in 1870, more ore than any one mine in the Marquette Region. It employed, during the winter of 1868 and during the summer, a somewhat less number of horses and mules. One animal moved about twelve tons per day, or 3,600 tons per year; but more than three-fourths of this stock was employed in getting "surface ore," a feature which does not exist in Marquette mining. The bluff (quarried) ore moved per horse employed, was more than five times the above amount. No steam-engine or locomotive was in use at the mine.

At the Caledonia and Keene mines, St. Lawrence County, New York, in 1869, three horses handled 27,500 tons of ore and waste, the average haul being over 700 feet all up grade, in places steep. This gives over 9,000 tons per head; steam was not employed for handling material at either mine.

The Sterling mine, Orange County, New York, shipped, in 1869, 40,000 tons of ore, which was handled under circumstances quite similar to those encountered in the Marquette Region, by two horses and one small stationary engine, which gives 20,000 tons per animal employed. The system of tramways and sidings at this mine is very complete.

Passing from American to Swedish mines, which are far deeper, and in which there is a larger percentage of rock mixed with ore, we find

that in the Presberg mines, in 1870, the total cost for handling ore and water drawing, was $14\frac{2}{3}$ per cent. of the whole cost, or 33 cents per ton of ore; and this amount included the handling of all the rock and other waste material, which in our table is embraced under *Dead-work.* If we take out of dead-work 10 cents for handling the waste material, and add it to the amount found above, we have 51 cents as total cost of handling Lake Superior ores, equal to twenty per cent. of the whole cost, or about fifty per cent. greater than in the Swedish mines, where steam and water are exclusively used.

5. *Machinery for Pumping and Hoisting.*—Notwithstanding the great cost of the work of horses, a large amount of machinery, as has already been remarked, is now in use, eight out of the eleven larger mines, having more or less complete plants. (1870.)

The introduction of machinery has so far seemed to make but little relative diminution in the number of horses employed, because of the greater amount of waste material which has to be moved in the later years. The figures given in the table opposite this item, $11\frac{2}{10}$ cents, is designed to be an approximation to the cost of running the machinery of such mines as have plants distributed over the entire product of those mines. I estimate that less than one-half of the product of such mines was handled by machinery in 1870. The actual cost of the ore so handled, including the *pumping*, varied from 14 to 21 cents, the mean as shown by my data being about 18 cents. This cost is made up of wages of engineers and firemen, say fifteen per cent.; fillers, landers and surface tramming sixty per cent.; fuel, repairs of machinery and supplies, say twenty-five per cent. This covers cost from miners' hands to cars or stock pile.

While this sum is materially less than the cost of the same work by horses, it is much greater than in the Copper Region of Lake Superior, where this business is brought to great perfection. Some of the appliances employed in the Copper Region can not be used at iron mines on account of the greater irregularity of the deposits. But time will introduce many economies, which will reduce this cost below the figures given. It must be borne in mind in comparing the cost of steam machinery with horses, that in the case of the engines all the pumping is included, while the horses handle only the ore. Making this correc-

tion, it is safe to say that it costs at least four times as much to handle the same ore by horses as by machinery.

The following described machinery plant has been recently erected at the New York mine, and will give a good idea of the kind in use. It consists of one steam engine with cylinder 16x24 inches, with bed cast solid in one piece: Valve is of the kind known as the H-valve, and is worked by link motion; steam pipe 4 inches in diameter, exhaust pipe 6 inches in diameter; engine supplied with the Judson governor. Pump for feeding boiler is worked from cross head. Main shaft is 5 inches in diameter, of hammered iron, and 6 feet long. One boiler 42 inch shell, 21 feet long, with two 14 in. flues. Smoke stack is 35 feet high and 20 inches in diameter. The winding drums are 4 feet in diameter, and of sufficient capacity to contain 525 feet of 1¼ inch wire rope. They are worked by positive clutch movement, thrown in and out of gear by means of hand wheels attached to lever by pinion and gear. The brakes are known as band brakes, which clamp the entire surface of the drum 5 inches in width, and are of sufficient power to hold a loaded skip at any point in case of accident. They are worked by levers with hand or foot, as may be desired. The drums make about 13½ revolutions per minute, the engine making 80, which gives the skip a speed of a trifle less than 3 feet per second. The skips are of heavy boiler iron, each having four 12 inch wheels. The capacity of each is 18 cubic feet, equal to about 1¼ tons of ore. The pump is 8 inches in diameter by 6 feet stroke, capable of discharging 243 gallons of water per minute. It is worked from a slotted crank arm, on end of main drum shaft, which admits of lengthening or shortening the stroke at pleasure. The pump is double acting with single valve on a new place. It is furnished with rods, travellers, connections, balance bobs, &c. This machinery was furnished complete in all its parts and set up at the mine in complete order for pumping and hoisting for $6,000, by the Iron Bay Foundry, Marquette, Mich., 1870.

V. *Management and General Expenses.*—This covers only those expenses in the mining region, and not salaries of officers above the superintendent, nor the cost of selling the ore.

1, 2.—*Salaries, Office Expenses, and Taxes.*—This element of cost constitutes less than 5 per cent. of the whole cost of the ore, amounting to

about 12 cents per ton. I am happy to note here a much better showing than in the Presberg Mines, Sweden, where this item, in 1870, cost 16½ per cent. of the whole, or 36 cents per ton of ore; nearly three times its cost with us. I presume the excess of this item in Sweden may be largely due to heavier taxes.

XXXV.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

At a Regular Meeting of the Society, January 17th, 1872.

APPARATUS FOR OBTAINING BORINGS BY DIRECT PRESSURE.

A paper read before the Society by Theodore Allen, C. E., Member of the Society.

The purpose for which this apparatus was designed was to enable borings—showing the true bottom, or hard bed of the river—to be obtained with greater rapidity than they could be made by the use of the usual boring-rod, especially where the nature of the work necessitated a floating machine.

It was found by the Engineers of the Dock Department of this city that, owing to the swell from passing vessels, to the eddies caused by the current in the slips, and to the rise and fall of the tide, the borings obtained by the ordinary means used in such cases, were not to be depended upon for accuracy, could be had with difficulty at the best, and occupied so long a time in making, that they decided to endeavor to obtain some apparatus by which the work could be more rapidly pushed.

At the suggestion of Mr. John D. Van Buren, Jr., a member of this Society, connected with the Department, I designed the machine herewith described, and I desire to acknowledge my indebtedness to Mr.

Van Buren, and also to Major Watson, of the same Department, for valuable suggestions in regard to the original design, and to the subsequent improvements thereon.

It occurred to me that direct pressure might be employed to force the tube through the overlying strata, from the fact that no difficulty had been experienced in driving tubes for water through gravel, sand and clay, and even through the softer species of rocks, and that that which had been accomplished by percussive force might be as readily accomplished by sustained pressure. Hydraulic pressure was chosen, as being the most readily controlled, easily managed, and most certain in its action.

As a basis, it was decided that a pressure equivalent to 1,000 pounds to the square inch upon the area of the tube would be ample for all purposes, that is, that no pile could be forced against a like resistance, and consequently when a medium adequate to support that pressure either from its own density, or from the friction and pressure of the superincumbent earth, was reached, a safe foundation upon which to build the docks would be obtained. To exert this pressure on the tube, the tube itself required to be very strong to resist bending, when unsupported in the water or thin mud. A wrought-iron tube, having an external diameter of $2\frac{7}{8}$ inches and an internal diameter of $1\frac{5}{8}$ inches, was selected. The tube was made in sections of 8 feet, the weight—about 150 pounds—of each piece being as great as could be easily handled. The tube was secured to a cross-head by a contrivance which, while holding the tube securely, would permit it at the same time to be turned around. At each end of this cross-head a piston rod was secured, the piston to which each rod was attached, moving vertically within a cast-iron cylinder of six inches diameter, and long enough to allow a stroke or movement slightly greater than the length of a section of the boring tube.

These cylinders were supplied by means of a steam-pump, of the type known as fly-wheel pumps.

A boiler of the vertical type was selected to supply steam for the pump.

A simple arrangement of pipes connected the pump with the hydraulic cylinders, and the attendant, by the action of a single lever, could instantly stop or reverse the movements of both pistons. A float or scow to carry this machinery had to be provided; it was desirable to have it as small as possible for facility of handling in crowded slips,

while at the same time there must be sufficient displacement to resist the pressure upon the tube and maintain great stability at all times. The larger the area of the water line, of course the less the scow would lift when the pressure was applied. The pressure to be resisted was as follows: area of boring tube $2.875^2 \times .7854 = 6.49$ square inches, $6.49 \times 1,000$ lbs. $= 6490$ lbs. total pressure.

Two feet draft of water was permitted, and it was thought that the scow might be allowed to rise 3 inches, by the resistance of the tube, without interfering with its stability; this gave a resistance per inch of draft of $\frac{6490}{3} = 2163.3$ lbs., or an area of water line of $\frac{2163.3}{5.33} = 405.8$ square feet. Making the scow twice the length of the beam, gave $\frac{405.8}{2} = 202.90$ square feet for one-half, or $\sqrt{202.90} = 14.24$ feet breadth of beam; taking into consideration the weight of the tube and connections, the scow was made 14 feet beam by 28 feet long. Wooden guides, faced with iron, were erected to secure the vertical movement of the piston rods, the tops of the guides being braced, as shown on the drawing.

A clamp to hold the tube when disconnected from the cross-head was secured to the deck, the base of the clamp frame forming a guide for the tube, and extending through the scow, so as to support the tube as far down as possible. This clamp could be removed, leaving a large opening through the scow, so that bent tubes might be drawn through. The arrangement of the clamp was such as to close of itself, remaining open and allowing the tube to pass only when the lever was held up by the attendant; thus the tube in being hoisted could only drop, if disconnected from the cross-head, by the carelessness of the attendant.

Water, to supply the cylinders and the boiler, was carried in tanks built in the scow, the cylinders when discharged emptying back into the tanks. The scow was also ballasted to trim her.

An anchor weighing 150 lbs., and 25 fathoms of chain cable, were secured to a windlass at each corner of the scow, so that the scow might be hauled into position in any direction, by slacking off on one side or end, and hauling up on the opposite side or end.

In the original design the tube was attached to a loose head passing through the cross-head, but at the suggestion of Major Watson an arrangement was designed by which the tube was allowed to pass through the cross-head, and was clamped below; by this means, when

there was needed a total length of not more than 32 to 48 feet of tube, the tube could be used without uncoupling; after the pistons had descended the full stroke, the tube was unclamped, the pistons raised, and at their full elevation the cross-head was again clamped to the tube.

In order to prevent one piston travelling faster than the other, and thus springing the tube, the piston rod at the cross-head end was made T shaped, and at the end of each arm of the T a hole was bored, somewhat larger than the bolt which passed through and secured them to the cross-head. By this means, if one piston should advance beyond the other, the strain from the inclination of the cross-head would be thrown on the outer arm of the T on the slower piston; and on the inner arm of the faster one, thus giving a leverage in favor of the slow piston; this was found to work very satisfactorily.

In order to regulate the pressure employed in forcing the boring tube down, a safety valve was provided having the usual weighted arm; by moving the weight any force desired could be brought to bear upon the tube; and during the test before the acceptance of the machine a pressure of 450 pounds to the square inch, in the hydraulic cylinders, was carried, equivalent to a total pressure downwards of 23,850 lbs. which would be 3,674 lbs. to each square inch of the boring tube.

In describing the means of connecting the tube with the cross-head it was stated that it was so devised as to permit the tube to be turned; the object of this was to enable the person in charge of the work to ascertain, when the further progress of the tube was checked, the character of the material against which it bore.

The lower end is provided with an auger bit forged of heavy steel, and a sample of the last ground entered generally adheres to the underside of the shoulder of the bit.

In operating the machine the cross-head is first raised to the extreme elevation, and as much of the tube is connected and passed through the guide-hole, as will enter without pressure; it is then further forced down until the strength of four men can force it no further by pulling upon it with their hands. At this depth it is considered that a pile will begin to hold. The cross-head is then clamped to the last length of the tube, and pressure applied; when the pistons have reached the bottom of the cylinders, the cross-head is disconnected, and raised—if the ground is soft, to its full height—if in hard

ground, to a height of about 5 feet,—and being again clamped, is forced down to the end of the stroke. The reason why the full stroke is not given in hard ground, is on account of the tendency of the tube to spring, and bind in the guide, when the pressure is great. The tube is also frequently turned around, as it is found the boring proceeds more easily when this is done.

Additional lengths are added, and this process is continued until the safety-valve lifts, showing that the requisite resisting material has been reached. The process is then reversed and the tube withdrawn. The safety-valve will sometimes lift before the depth at which it has been expected the tube would stop, has been reached; this generally is caused by boulders. If after turning the tube and applying the pressure several times, the tube will not work clear, it is withdrawn, and the position of the scow slightly altered; when the tube is again driven down, two or three soundings will suffice to show the character of the obstruction. In some cases, the tenacity of the soil is such, that the scow will be drawn down by the attempt to withdraw the tube until the deck is at the level of the water; the continued strain caused by this immersion, after a short time, is sufficient to overcome the difficulty.

In 50 feet of mud, sand, and gravel, the average time occupied is 45 minutes, which includes withdrawing, uncoupling, and recording. Ordinarily 7 to 8 soundings are taken in 6 hours. The greatest length of tube, so far used, has been 114 feet, and the greatest depth penetrated has been 87 feet 10 inches.

The machine has been in actual operation over a year, and during that period over 1,500 borings have been made; nearly 600 from Sept. 1st to Dec. 31st, 1871. It has been successfully worked in 60 feet of water, and has been used in all weathers.

It is not claimed for this machine, that the strata can be so accurately defined as by the usual boring-rod, where the apparatus can be stationed upon a firm foundation; but it is claimed that, where borings are needed to establish the depth to which piles must be driven, or foundations carried down for piers of wharves, bridges, or other structures, all the data necessary can be obtained by this machine in much less time, and at far less cost, than by the use of boring-rods. The amount of pressure required, and the sound transmitted through the tube when turned, show, with considerable accuracy, the nature and extent of the strata through which the tube is being forced, as shown in the profile of

soundings south of Pier 1, North River, where, overlying the rock, is a bed of stiff mud, over which is a layer of gravel, and above this a later deposit of mud. The point at which the tube met the surface of these various deposits was shown by the difference in the pressure required, and the depth at once read off and recorded.

Mr. Allen's paper elicited the following discussion:

Mr. Morse: Is the tube open ?

Mr. Allen: It is closed at lower end ; at first a loose sleeve on auger was tried; in the sleeve were drilled many holes, into which, it was thought, the earth would pack; but the plan did not work well; the sleeve split. The Department of Docks did not care for samples of the soil. The nature of the soil can be judged by the pressure required to puncture it, and by turning the rod.

Mr. Morse: This can be done with a common rod; by the feeling, can distinguish between sand and mud.

Mr. Constable: What is meant by the red line in the profile ?

Mr. Allen: That is the base of foundation, below which are to be piles; the tube described was $2\frac{1}{8}''$ diameter; it has been proposed to use a larger tube, and have a drill inside.

Mr. Macdonald: Could not you have an open tube with a circular cutting edge, and bring up the contents forced into it ?

Mr. Allen: The tube must be stiff, and it is difficult to make proper joints.

Mr. Macdonald: Suppose you had a sharp steel edge, and cut a cylinder of material as you forced the tube down ?

Mr. Allen: It would be almost impossible to withdraw such a tube.

Mr. Morse: This plan is identical with the State system of sinking salt wells; the tube is weighted; a crab is coupled to it; have 6" pipe (the ordinary tubes sold in market) with outside connections and steel cutting edge, smooth inside. The tube is grasped outside and pushed down. Put down 350 ft., soft mud first, then layers of gravel cemented with gypsum. The diameter at edge is slightly larger than of couplings. When strike hard strata, will generally break through; if not, a tool inserted in tube, to drill through; a contrivance is also used to swell the hole; in the building is rigged a pile-driver, to jar and drive the tube when stuck. When the tube is drawn out, the contents remain in.

Mr. Allen: The connections are made with studs welded on to one

pipe, which tube into groove of sleeve on the end of the other (Look at drawing); the tube now used is $\frac{5}{8}''$ thick. At 85th street lately had 102′ of tube in ground, an ice field floated down and dragged the scow; the tube was bent at the scow and the ground.

Mr. Morse : A larger tube, with outside couplings and butt joints could be used.

Mr. Allen : In stiff mud, the scow is pulled down to surface of water.

Mr. Collingwood : What is the displacement of the scow ?

Mr. Allen : I mentioned that in the paper. At the New York side, East River Bridge, there was no difficulty in pulling up a 6″ tube with a 10 ton jack, from a depth of 55′ in the ground, or 92′ below high water.

Mr. Allen : Had 12 tons to draw with. The Department of Docks will build a larger machine, to work in and around slips.

Mr. Morse : I saw a lot of 4″ pipe, with boring apparatus, on the dock ; what was that used for ?

Mr. Collingwood : That was used by the Department of Docks for making soundings before the machine designed by Mr. Allen was in operation.

Mr. Morse : I have prepared tubing for 50 to 100 oil wells. This is quite different.

Mr. Allen : There they drive tube through rock.

Mr. Morris : The first tube is of cast iron, driven with a hammer, and is not cleared out.

Mr. Morse asked about splitting the sleeve.

Mr. Allen : 1st pipe used was $\frac{3}{8}''$ thick, then $\frac{1}{2}''$, and now $\frac{5}{8}''$, the joints break.

Mr. Ward : Any one upon trial will be satisfied that it is difficult to drive sand from a few feet in length of a 3″ pipe.

Mr. Collingwood : I found sometimes that in driving the tubes at our work, a small boulder would be driven in advance of the pipe for a depth of 5′ to 10′, so that stratification could not be accurately determined.

Mr. Allen : By a nice arrangement of gauges, the nature of the soil could be known ; variation of pressure, from 100 to 300 lbs. per square inch would tell.

Mr. Morse : In the Savannah swamps a long sounding-rod was

used. When pushed down into the mud, a sand stratum ½ inch thick could be told by hearing and feeling ; on sand the rod would stop ; if it came against a cypress root, the resistance would be great ; by feeling could tell this from a hard stratum, but not by pressure.

Mr. Allen : It could be known at once by pressure, that something was met ; a number of soundings near by, would show its extent ; the gauge going up would reveal strata.

Mr. Collingwood : How does this method differ from the German ; they have all sorts of augers to work inside of tubes, scoops, &c.

Mr. Allen : The method of sinking is different, in the direct application of hydraulic pressure.

Mr. Morse : Can any one here say whether the plan described of sinking the Syracuse salt wells, originated with Judge Geddes or not?

Mr. Ward : Pile drivers here take preliminary soundings in substantially the same way ; they use a tube and cross-tree lashed fast.

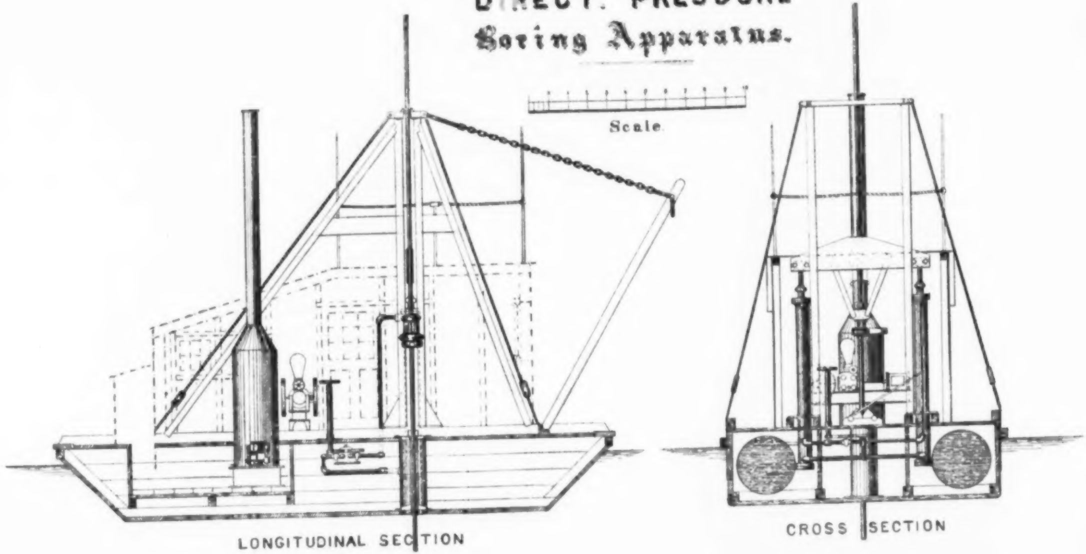
Mr. Morse: The peculiarity of the system mentioned is, having a heavy weight upon a platform clamped to force down the tube.

Mr. Morris : It is the usual practice in sinking artesian wells.

Mr. Morse : Salt wells were sunk before the Revolutionary War.

Col. Sedgwick : The object, I take it, of this machine, is not to detect strata, but to determine a foundation capable of sustaining a given pressure, one sufficient to bear piles.

DIRECT. PRESSURE
Boring Apparatus.



XXXVI.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT A REGULAR MEETING OF THE SOCIETY, DECEMBER 6TH, 1871.

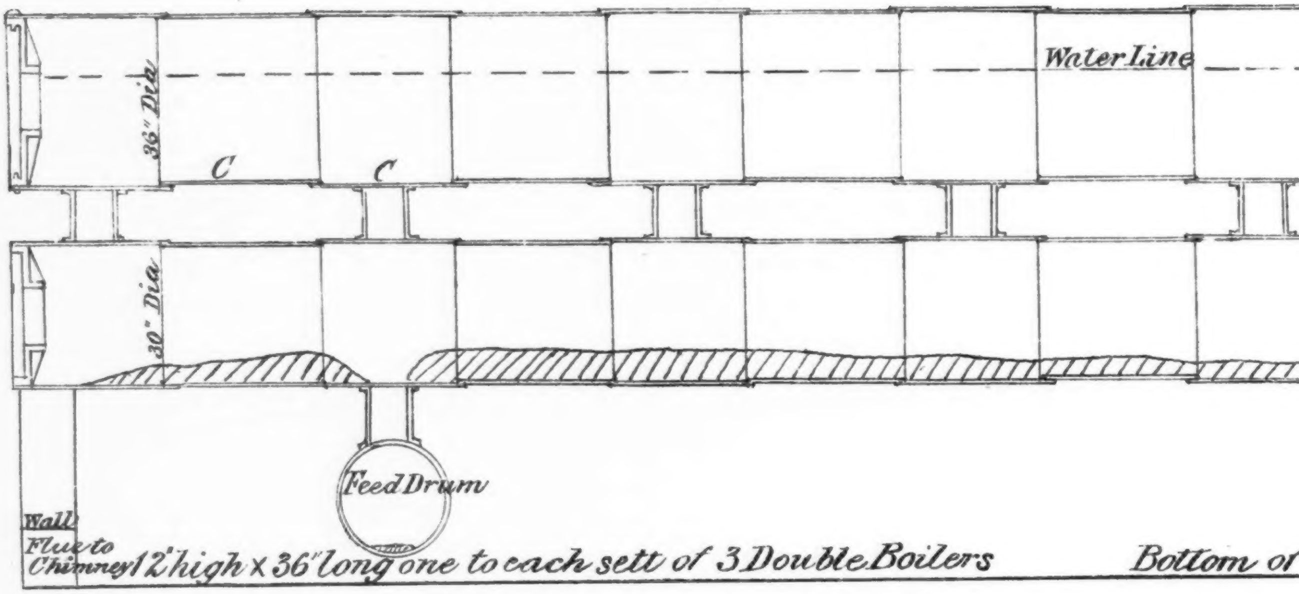
ON THE USE OF A SURFACE CONDENSER IN CONNECTION WITH A SET OF BLAST FURNACE BOILERS, AT THE FRANKLIN IRON WORKS. ONEIDA CO., N. Y.

A paper read by W. B. COGSWELL, C. E., Member of the Society.

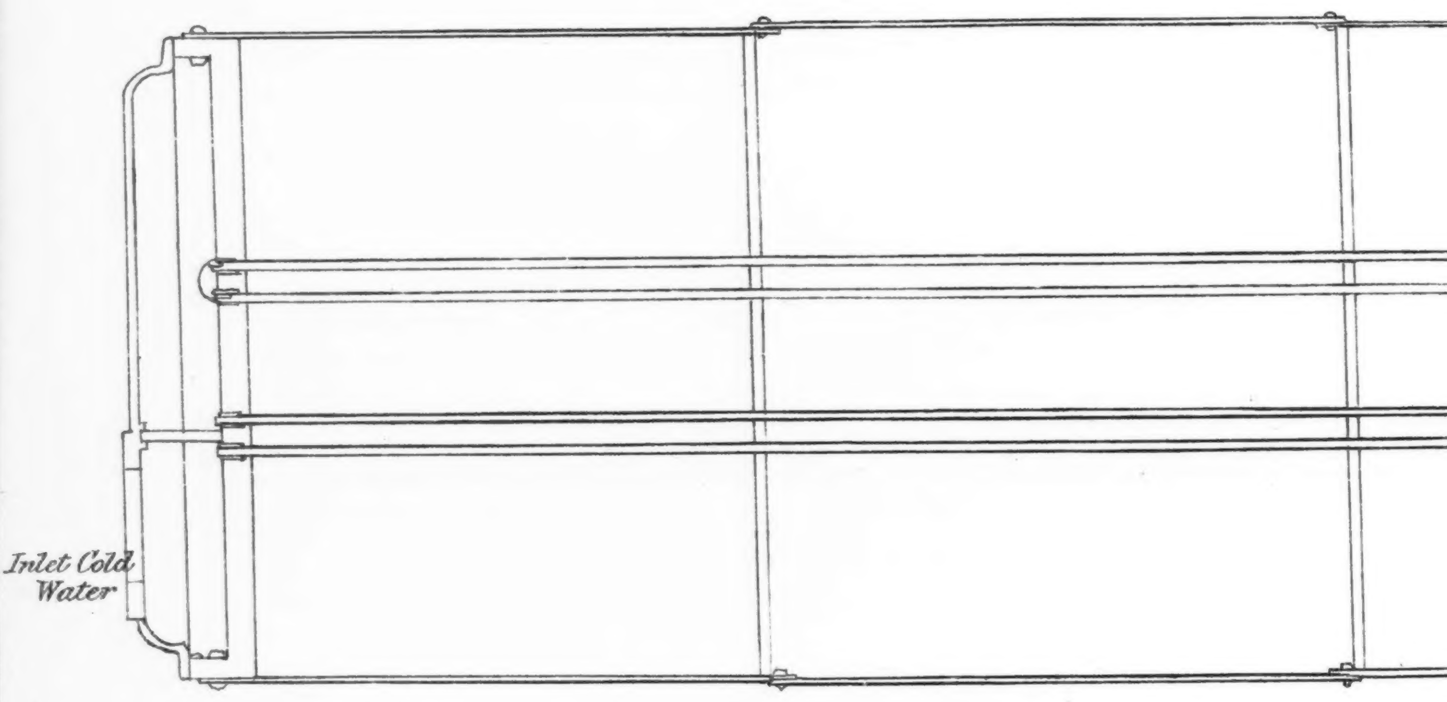
The water in this region containing nearly as much of the carbonate of lime as can be held in solution, and the long continued service under steam of a set of blast furnace boilers, suggested the use of a surface condenser, to prevent in a measure the large deposit of scale in using the natural water. The condenser consists of two cast-iron heads, to cover the water space at the ends, two cast-iron tube-heads, and a wrought-iron casing. There are 860 tubes 10 feet long, ¾ inch diameter, No. 20 W. G. in thickness. The circulating, or cooling water passes through the tubes, and three times through the length of the condenser. Opposite the

steam inlet at E, in accompanying sketch, is riveted a wrought-iron plate to equalize the distribution of the steam throughout the length of the condenser. There are about 1,600 square feet of cooling surface, and 2,800 square feet of heating surface to the boilers. The cooling surface is large in proportion to the heating surface, but was purposely made so, that it might remain sufficient when the tubes become covered with grease and a kind of slime that comes from the water, and which I have never seen to so great an extent in salt water. The engine used is 38 inches diameter of cylinder, and 10-foot stroke, without cut-off, or balance wheels, or crank, working from 100 to 120 feet per minute. It was found that, if the circulating water was allowed to become over 100 deg. F., a scale was deposited upon the inside of the tubes. A sample of this scale is sent. By increasing the quantity of circulating water, which can be adjusted by valves, so that it passes away from the condenser at nearly the temperature of that at the inlet, this difficulty was prevented. There are a pair of hoisting engines, 7 inch by 12 inch cylinder, not connected with the condenser. The waste of the steam used here, as well as that at the safety valves, which it is impossible to perfectly control, is supplied from the natural water. After having been under steam for 10 months, one set of boilers were opened and examined. In the upper boiler, at B B B in the sketch, the scale was $\frac{3}{32}$ inch thick. A sample is marked B. This deposit gradually diminished in thickness, until at the points C C it was no thicker than an ordinary visiting card. In the lower boiler there was a large deposit. At the point "A" it was 5 inches thick; this diminished as shown in the sketch. When the boiler was opened, it was soft like mud, and was very easily removed. There was none of the scale like sample "B" in this boiler. The sample of material marked "D" was found in the upper boiler, and has the appearance of having floated upon the surface. It probably came from the oil (sperm) used, which is reduced to the lowest possible quantity, and supplied by an oil-cup whose feed is continuous. The condensed steam is drawn from the condenser into a large tank, to allow the oil to rise to the top, and so be taken off. The condenser can be cut off from the engine for repairs. The tube heads are made and packed under Wm. A. Lighthall's patents. Without the condenser it would be necessary to clean the boilers every two months. This would reduce the quantity of

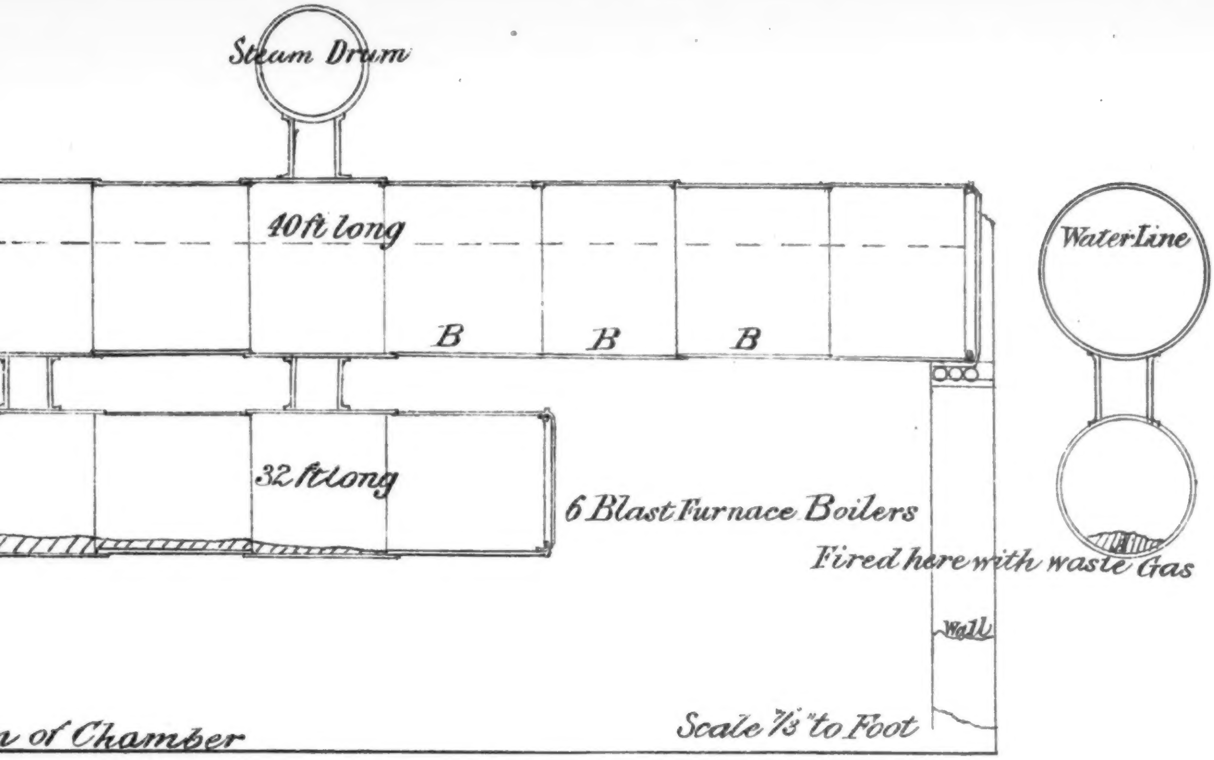
iron made for that week some 50 tons; the profit at $6 per ton that would be lost in a year would be $1,800. The boilers would have been injured by one year's run. Supposing the cleaning to cost the same, this would leave a balance of $1,500 in favor of using the condenser, the cost of which was less than $2,000. The vacuum is not used generally, but can be when desired, as a vacuum pump is connected. The circulating water is obtained from an abandoned water power, which has about 16 feet head. The supply pipe is 8 inches diameter.

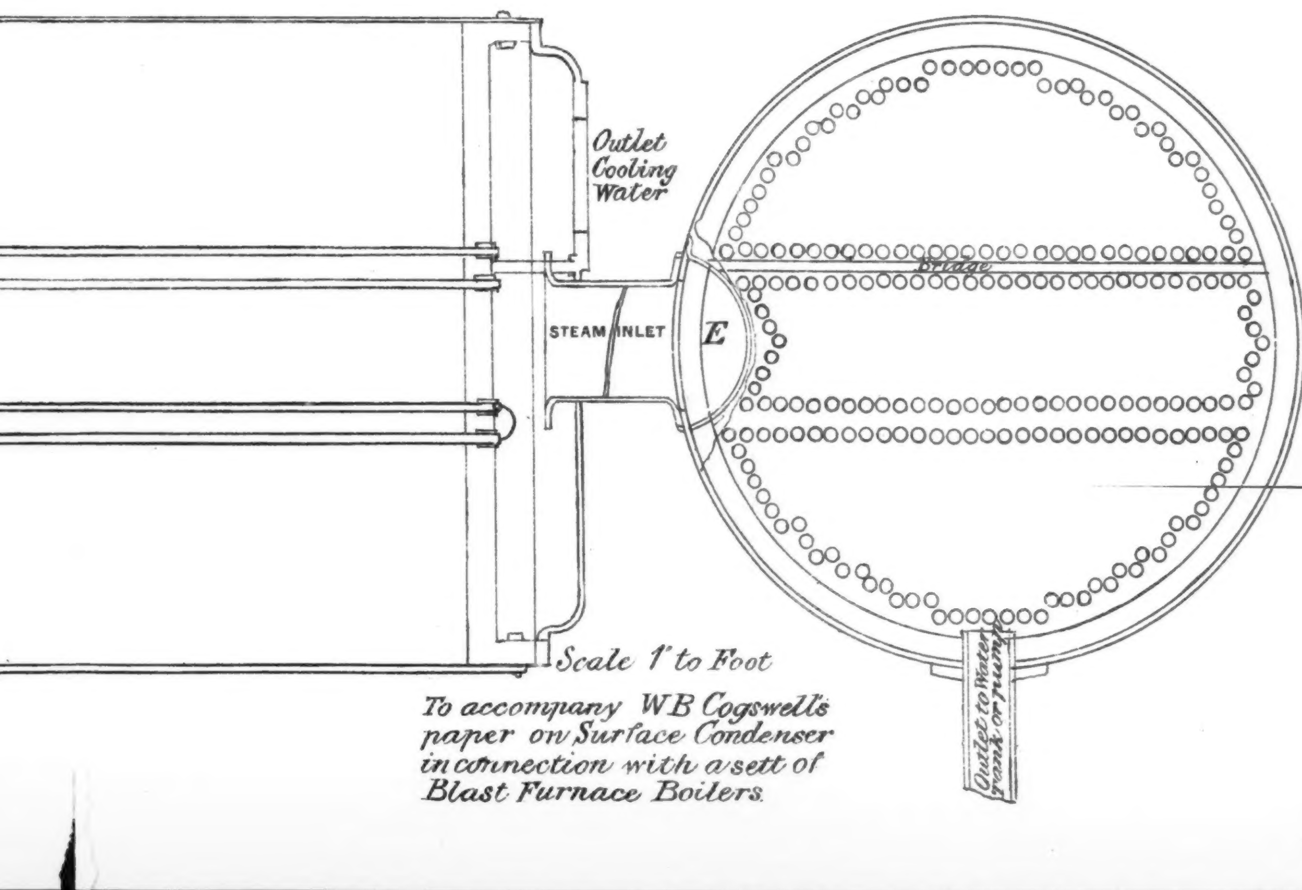


Water Line
36" Dia
C
C
30" Dia
Feed Drum
Wall
Flue to Chimney 12" high x 36" long one to each sett of 3 Double Boilers
Bottom of



Inlet Cold Water





To accompany W.B. Cogswell's
paper on Surface Condenser
in connection with a sett of
Blast Furnace Boilers.

XXXVII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

AT A REGULAR MEETING OF THE SOCIETY, MARCH 6TH, 1872.

MEMORANDUM AND TABLES, EXHIBITING THE RESULTS OF SOME OF DARCY'S EXPERIMENTS ON THE FLOW OF WATER THROUGH PIPES.

A paper presented by JAMES B. FRANCIS, C. E., Member of the Society.

The experiments of Henry Darcy, on the flow of water in pipes, made in the years 1850 and 1851, are given in detail in his work, published in Paris in 1857, entitled, *Recherches expérimentales relatives au mouvement de l' eau dans les tuyaux.* While making the experiments, Darcy occupied the position of Director of the municipal service of the city of Paris, the duties of which office included the distribution of the water supply, giving him great facilities for studying the laws governing the motion of water in pipes. His experiments were made on a scale and with a completeness of apparatus far exceeding any others that have been published.

The experiments, as published by Darcy, are referred to the metre as the unit, which is not a convenient form for study by the American Engineer. Having had occasion to make use of the experiments on

new cast-iron pipes, and on those which from long service had become lined with deposit, I have had them reduced to English measures; and it having been suggested to me that in this form they might be useful to other members of the profession, I offer them to the Society for publication.

Description of Table I.—Experiments on New Cast-Iron Pipes. —Most of the columns are sufficiently explained in their respective headings. Column 11 is computed by Darcy's formula for new cast-iron pipes, slightly modified for convenience. His formula, as given at page 228 of his work, above cited, when reduced to English measures, may be put under the form

$$i = (0.00370874\ d + 0.00372672)\frac{V^2}{d^2}$$

in which

i = the slope or loss of head per foot in length of pipe, in feet.

V = the mean velocity in the pipe, in feet per second.

d = the diameter of the pipe, in inches.

Without sensibly affecting results, this formula may be put in the simpler form

$$i = 0.00371\ (d + 1)\frac{V^2}{d^2}$$

Column 12 gives the proportional difference between the result of experiment in column 6, and of computation by this formula in column 11. In the experiments, in which the velocity is less than one foot per second, the mean proportional difference is — 0.23481, indicating that the formula gives the values of i in these experiments on an average about 23 per cent. less than was found by experiment. In the experiments in which the velocity is greater than one foot per second, the mean proportional difference, disregarding the signs, is 0.0744, indicating that the formula gives values of i differing, in the average, about 7½ per cent. from the experiments.

Column 13 gives the value of the coefficient, determined separately from each experiment. Its mean value, in the experiments in which the velocity is less than one foot per second, is 0.004988. The mean, in the experiments in which the velocity is greater than one foot per second, is 0.003716, being very nearly the same as in the formula in column 11, which was deduced directly from Darcy's formula.

Table II.—Experiments on Old Cast-Iron Pipes, Lined with Deposit.—Darcy gives no definite description of the kind of deposit

with which the pipes he experimented on were lined, but by a comparison of several passages in his work, it is inferred that it was calcareous, and did not present a marked tubercular character. He deduced the mean diameters of the pipes from the volumes of water they contained. This was done both before and after the deposit was removed. In the latter case, the diameter may be considered the same as it was when the pipe was new. In the practical application of any formula deduced from the experiments, the latter, or original diameter, is the only one that can conveniently be used. From the diameters given by Darcy of the three foul pipes on which experiments were made, I infer that the mean thickness of the deposits were 0.0098, 0.0118 and 0.0295 inch respectively, averaging, say, $\frac{1}{50}$ of an inch. I have no similar measures of the amount of deposit found in the old pipes in any of our Water Works, but I think it is usually much greater than the above. The great variation in the deposit in iron pipes, not only in the amount, but in the roughness of the surface it presents to the water, on which latter characteristic the resistance largely depends, must, generally, prevent all pretension to precision in the application of any formula to foul pipes.

Column 15 is computed by Darcy's formula for old cast-iron pipes, reduced to English measures, it being the same as the formula for new cast-iron pipes, except that the coefficient is twice as great, and consequently giving values of i twice as great for old as for new pipes. In this formula, the diameter and velocity correspond to the capacity of the pipe as reduced in section by the deposit. To find the coefficient in a similar formula, in which the diameter and velocity refer to the pipe when new and clean, its value, as determined from each experiment separately, is given in column 17. The mean of all the values of c, thus determined, is 0.008402. The mean in the experiments in which the velocity is greater than one foot per second is 0.008166. Column 18 is computed with this value of the coefficient, taken to the nearest fourth decimal. Column 19 gives the proportional differences between the results of experiment as given in column 9 and by the formula in column 18. Omitting the experiments in which the velocity is less than one foot per second, the mean proportional difference, disregarding the signs, is 0.03424, indicating that the formula gives results differing, on an average, about $3\frac{1}{2}$ per cent. from the experiments.

TABLE I.

Experiments by H. Darcy on the flow of water through New Cast-Iron Pipes, reduced to English measures.

1	2	3	4	5	6	7	8	9	10	11	12	13
Number of the Experiment.	Interior diameter of the pipe.		Sectional area of the pipe.	Head, by experiment, to overcome the resistance of 100 metres = 328.09 feet in length of straight pipe.	Slope, or loss of head, per foot in length of pipe i.	Quantity of water discharged during the experiment.	Time in which the experimental discharge was made.	Quantity of water discharged by experiment.	Mean velocity in the pipe by experiment V.	Value of i by the formula $i = 0.00371\,(d+1)\,\frac{V^2}{d^2}$.	Proportional difference of the value of i by experiment and by the formula in the preceding column	Value of C in the formula $i = C(d+1)\frac{V^2}{d^2}$ deduced from experiment.
	Feet.	d. Inches.	Square feet.	Feet.	Feet.	Cubic feet.	Sec.	Cb. ft. per sec.	Ft. per sec.	Feet.		
134	0.2687	3.2245	0.056708	0.066	0.00020	5.889	360.	0.0164	0.2885	0.00013	— 0.35000	0.005916
135	"	"	"	0.272	0.00083	49.362	1548.	0.0319	0.5623	0.00048	— 0.42169	0.006460
136	"	"	"	0.761	0.00232	55.881	840.	0.0665	1.1732	0.00207	— 0.10776	0.004149
137	"	"	"	1.742	0.00531	56.341	540.	0.1043	1.8399	0.00510	— 0.03955	0.003860
138	"	"	"	3.347	0.01020	52.966	360.	0.1471	2.5946	0.01015	— 0.00490	0.003729
139	"	"	"	7.398	0.02255	56.239	255.	0.2205	3.8892	0.02280	+ 0.01109	0.003669
140	"	"	"	10.525	0.03208	475.005	1800.	0.2639	4.6537	0.03265	+ 0.01777	0.003646
141	"	"	"	13.260	0.04041	505.315	1730.	0.2921	5.1509	0.04000	— 0.01027	0.003749
142	"	"	"	31.323	0.09547	711.954	1560.	0.4564	8.0482	0.09764	+ 0.02273	0.003627
143	"	"	"	32.494	0.09904	694.155	1500.	0.4628	8.1609	0.10039	+ 0.01363	0.003660
144	"	"	"	39.299	0.11978	698.357	1380.	0.5061	8.9242	0.12005	+ 0.00225	0.003702
145	"	"	"	55.142	0.16807	*	900.					
146	"	"	"	56.011	0.17072	728.884	1200.	0.6074	10.7115	0.17296	+ 0.01312	0.003662
147	0.4501	5.4017	0.159142	0.679	0.00024	55.983	720.	0.0778	0.4887	0.00019	— 0.20833	0.004581
148	"	"	"	0.285	0.00087	56.008	360.	0.1556	0.977[illegible]	0.00078	— 0.10345	0.004148
149	"	"	"	0.686	0.00209	45.885	180.	0.2549	1.6021	0.00209	0.	0.003711
150	"	"	"	1.558	0.00475	716.603	1800.	0.3981	2.5021	0.00510	+ 0.07368	0.003458
151	"	"	"	4.134	0.01260	680.760	1020.	0.6674	4.1946	0.01432	+ 0.13651	0.003264
152	"	"	"	7.300	0.02225	751.171	840.	0.8943	5.6202	0.02571	+ 0.15551	0.003211
153	"	"	"	10.886	0.03318	722.519	660.	1.0947	6.8801	0.03853	+ 0.16124	0.003195
154	"	"	"	12.812	0.03905	714.117	600.	1.1902	7.4801	0.04554	+ 0.16620	0.003181
155	"	"	"	32.323	0.09852	3305.385	1740.	1.8996	11.9389	0.11602	+ 0.17763	0.003150
156	"	"	"	54.975	0.16756	5143.369	2100.	2.4492	15.3929	0.19287	+ 0.15105	0.003223

157	0.6155	7.3860	0.297537	0.089	0.00027	373.835	1920.	0.1947	0.6544	0.00024	— 0.11111	0.004102
158	"	"	"	0.574	0.00175	525.825	1080.	0.4869	1.6363	0.00153	— 0.12571	0.004252
159	"	"	"	1.207	0.00368	624.616	840.	0.7436	2.4991	0.00356	— 0.03261	0.003833
160	"	"	"	2.641	0.00805	596.983	540.	1.1055	3.7155	0.00787	— 0.02236	0.003793
161	"	"	"	4.396	0.01340	3502.288	2400.	1.4593	4.9045	0.01372	+ 0.02388	0.003624
162	"	"	"	7.382	0.02250	6139.694	3240.	1.8950	6.3688	0.02313	+ 0.02800	0.003609
163	"	"	"	12.500	0.03810	4127.107	1680.	2.4566	8.2564	0.03888	+ 0.02047	0.003636
164	"	"	"	36.024	0.10980	5841.725	1380.	4.2331	14.2271	0.11544	+ 0.05137	0.003529
165	"	"	"	47.872	0.14591	7246.293	1500.	4.8309	16.2360	0.15034	+ 0.03036	0.003601
182	0.9751	11.7010	0.746747	0.092	0.00028	716.631	1200.	0.5972	0.7997	0.00022	— 0.21429	0.004720
183	"	"	"	0.390	0.00119	552.794	420.	1.3162	1.7625	0.00107	— 0.10084	0.004130
184	"	"	"	0.883	0.00269	5470.996	2700.	2.0263	2.7134	0.00253	— 0.05948	0.003[illegible]39
185	"	"	"	1.762	0.00537	3732.263	1320.	2.8275	3.7863	0.00493	— 0.08194	0.004038
186	"	"	"	3.625	0.01105	3874.075	9[illegible]0.	4.0355	5.4039	0.01005	— 0.09[illegible]50	0.004079
187	"	"	"	7.562	0.02305	4211.629	720.	5.8495	7.8330	0.02112	— 0.08373	0.004050
188	"	"	"	10.515	0.03205	2774.557	405.	6.8508	9.1738	0.02896	— 0.09641	0.004105
189	"	"	"	13.473	0.04106	5568.976	720.	7.7347	10.3575	0.03692	— 0.10094	0.004126
190	1.6427	19.7130	2.119498	0.148	0.00045	2625.551	900.	2.9173	1.3765	0.00037	— 0.17778	0.004456
191	"	"	"	0.148	0.00045	2801.115	900.	3.1123	1.4685	0.00043	— 0.04444	0.003915
192	"	"	"	0.197	0.00060	7315.822	2220.	3.2954	1.5549	0.00048	— 0.20000	0.004656
193	"	"	"	0.394	0.00120	4290.421	780.	5.5005	2.5954	0.00133	+ 0.10833	0.003342
194	"	"	"	0.410	0.00125	6616.085	1200.	5.5134	2.6014	0.00134	+ 0.07200	0.003465
195	"	"	"	0.689	0.00210	†						
196	"	"	"	0.755	0.00230	6022.750	780.	7.7215	3.6433	0.00262	+ 0.13913	0.003251
197	"	"	"	0.853	0.00260	7454.242	960.	7.7648	3.6637	0.00265	+ 0.01923	0.003634
198	"	"	"	0.820	0.00250	6100.008	780.	7.8205	3.6900	0.00269	+ 0.07600	0.003445

* Not given by Darcy. † Apparently erroneously given by Darcy.

TABLE II.

Experiments by H. Darcy, on the flow of water through Old Cast-Iron Pipes, lined with deposit, reduced to English Measures.

1	2	3	4	5	6	7	8	9	10	11
Number of the experiment.	Interior diameter of the pipe, before the deposit was removed.		Sectional area of the pipe, before the deposit was removed.	Interior diameter of the pipe, after the deposit was removed.		Sectional area of the pipe, after the deposit was removed.	Head, by experiment, to overcome the resistance of 100 metres = 328.09 ft., in length of straight pipe.	Slope, or loss of head, per foot in length of pipe.	Quantity of water discharged during the experiment.	Time in which the experimental discharge was made.
		d'.			d.			i		
	Feet.	Inches.	Square ft.	Feet.	Inches.	Square ft.	Feet.	Feet.	Cubic feet.	Seconds.
108	0.1178	1.4134	0.010896	0.1194	1.4331	0.011202	0.082	0.00025	*	360.
109	"	"	"	"	"	"	0.233	0.00071	6.283	2160.
110	"	"	"	"	"	"	0.600	0.00183	4.190	900.
111	"	"	"	"	"	"	2.198	0.00670	5.420	600.
112	"	"	"	"	"	"	5.003	0.01525	4.901	360.
113	"	"	"	"	"	"	10.630	0.03240	5.910	300.
114	"	"	"	"	"	"	13.632	0.04155	5.432	240.
122	0.2608	3.1300	0.053433	0.2628	3.1536	0.054243	0.213	0.00065	19.428	900.
123	"	"	"	"	"	"	0.820	0.00250	29.065	660.
124	"	"	"	"	"	"	2.379	0.00725	42.256	540.
125	"	"	"	"	"	"	5.282	0.01610	57.005	480.
126	"	"	"	"	"	"	10.171	0.03100	53.810	330.
127	"	"	"	"	"	"	14.879	0.04535	57.005	285.
166	0.7979	9.5750	0.500038	0.8028	9.6340	0.506225	0.308	0.00094	665.352	1320.
167	"	"	"	"	"	"	0.663	0.00202	578.192	780.
168	"	"	"	"	"	"	1.552	0.00473	2089.776	1800.
169	"	"	"	"	"	"	3.773	0.01150	3592.331	1980.
170	"	"	"	"	"	"	7.513	0.02290	4261.409	1680.
171	"	"	"	"	"	"	10.499	0.03200	7040.364	2340.
172	"	"	"	"	"	"	13.468	0.04105	7552.717	2220.
173	"	"	"	"	"	"	45.870	0.13981	7547.338	1200.

* Apparently erroneously given by Darcy.

TABLE II—(*Continued.*)

Experiments by H. Darcy, on the flow of water through Old Cast-Iron Pipes, lined with deposit, reduced to English Measures.

12	13	14	15	16	17	18	19
Quantity of water discharged by experiment.	Mean velocity in the pipe by experiment. Before the deposit was removed. V'.	Mean velocity in the pipe by experiment. Supposing the deposit to be removed. V.	Value of i, computed by the formula, $i = 0.00742\,(d' + 1)\frac{V'^2}{d'^2}$	Proportional difference of the value of i by experiment, and by the formula in the preceding column.	Value of C in the formula, $i = C\,(d + 1)\frac{V^2}{d^2}$ deduced from the experiment.	Value of i, computed by the formula, $i = 0.0082\,(d + 1)\frac{V^2}{d^2}$.	Proportional difference of the values of i by experiment, and by the formula in the preceding column.
Cubic feet per second.	Feet per sec.	Feet per sec.	Feet.			Feet.	
0.0029	0.2669	0.2597	0.00064	−0.09859	0.008889	0.00065	−0.08451
0.0047	0.4273	0.4157	0.00164	−0.10383	0.008941	0.00168	−0.08197
0.0090	0.8291	0.8064	0.00616	−0.08060	0.008696	0.00632	−0.05672
0.0136	1.2494	1.2153	0.01399	−0.08262	0.008716	0.01435	−0.05902
0.0197	1.8079	1.7585	0.02930	−0.09768	0.008844	0.03004	−0.07284
0.0226	2.0772	2.0205	0.03868	−0.06907	0.008591	0.03966	−0.04549
0.0216	0.4040	0.3980	0.00051	−0.21538	0.009827	0.00054	−0.16923
0.0440	0.8242	0.8119	0.00212	−0.15200	0.009081	0.00226	−0.09600
0.0783	1.4645	1.4426	0.00671	−0.07448	0.008341	0.00713	−0.01655
0.1188	2.2226	2.1894	0.01545	−0.04037	0.008042	0.01642	+0.01988
0.1631	3.0517	3.0061	0.02913	−0.06032	0.008214	0.03095	−0.00161
0.2000	3.7434	3.6875	0.04383	−0.03352	0.007986	0.04657	+0.02690
0.5041	1.0080	0.9957	0.00087	−0.07447	0.008275	0.00093	−0.01064
0.7413	1.4824	1.4643	0.00188	−0.06931	0.008222	0.00201	−0.00495
1.1610	2 3218	2.2934	0.00461	−0.02537	0.007849	0.00494	+0.04440
1.8143	3.6283	3.5840	0.01127	−0.02000	0.007814	0.01207	+0.04957
2.5366	5.0727	5.0107	0.02202	−0.03843	0.007961	0.02359	+0.03013
3.0087	6.0169	5.9434	0.03099	−0.03156	0.007907	0.03319	+0.03719
3.4021	6.8037	6.7206	0.03962	−0.03484	0.007933	0.04243	+0.03362
6.2894	12.5779	12.4242	0.13540	−0.03154	0.007905	0.14502	+0.03726

XXXVIII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

EARLY HISTORY OF RAILWAYS AND ORIGIN OF GAUGE.

A paper read by J. DUTTON STEELE, Civil Engineer, Member of the Society.

Railways are as old as civilization; stone ways, adapted to the passage of wheeled carriages, are still in existence in the vicinity of the Egyptian quarries, whence the massive stones were taken to build the Pyramids. Wooden railways have been used in the mining districts of Germany from time immemorial; they were introduced into England early in the seventeenth century, and to that country we are chiefly indebted for their improvement, and the development into that system which has become the life of commerce.

We find in the History of Coal Mining at Newcastle-upon-the-Tyne, as early as 1602, a complaint, in quaint old English language, of the increasing badness of the roads, by which the loads of carts were being reduced; and in 1649, an account of a Mr. Beaumont, who expended £30,000 in improving the ways and wagons, and in introducing rare engines for boring with iron rods, to try the thickness of the veins, and for removing water from the pits. In 1676, the ways are described as rails of timber, straight and parallel, on which bulky carts on four rollers, are moved with such ease that a horse will draw four or five

tons ; but not until 1765 do we hear of something like a regularly constructed railway, which was graded by cutting and filling and had wooden rails seven inches square, resting upon cross sleepers four feet apart. We then have an improvement on this wagon-way, called a "double way," by which the number of cross-sleepers are doubled, and the rails are also doubled by placing one on the top of the other, with cinder packed between them. In 1770, the railway is described as a great work, carried over irregular ground, a distance of nine or ten miles, with wooden rails rounded on the top, and wheels made of cast iron, hollowed to fit the rounded surface. Then we have the rejection of the outer flanges of the metal wheels, the tread made flat, and iron placed on the curves to check the wear of the rails. In 1776, we have a cast-iron railway built in Sheffield, with flanges, projecting upwards, cast upon the rail, and both flanges on the wheels dispensed with ; this was too great an improvement for the temper of the workpeople, who got up a riot and destroyed it.

Edge rails were introduced in 1789, which again restored the inside flange to the wheels, and, in 1800, Benjamin Ostram made the improvement of supporting the ends of the rails with stone instead of wood, which gave them the name of "Ostram roads," afterwards abbreviated to "Tram roads." Malleable iron rails were introduced in 1808.

All these roads had a gauge of four feet eight and a half inches, which was the gauge of the ordinary road wagons to which they were originally adapted, and with which, from their slow progress, they long continued to be closely allied.

One hundred and fifty years of English experience had been required to advance them from timbers laid down to prevent the cart wheels from sinking in the mud to an iron rail and iron wheels, somewhat like the mode of the present day. Whilst Beaumont was expending his £30,000 at Newcastle, De Cans lay incarcerated in a madhouse at Paris for conceiving the idea that carriages could be moved by steam, calling to the people as they passed, "I am not mad ! I am not mad ! but have made a discovery !" The idea was expanded by James Watt in 1759, by Oliver Evans in 1772, and by William Symington in 1784.

The year 1800 found Tram roads so popular that they were talked of for general traffic ; the Duke of Bridgewater saw "danger in them;" George Stephenson who had risen to the charge of a hoisting engine at the mouth of a coal pit, at the age of twenty, was slowly but surely developing.

We now find attention strongly turned to steam locomotion; in 1802 an attempt was made to introduce it on common roads, and in 1804 upon railroads, and the efforts continued with but little success until 1814, when the discovery was made that there was adhesion enough between the wheels and rails to produce motion, and cogs chains and similar cumbrous contrivances were rejected.

It is remarkable how the mechanical mind clung to the idea that the wheels of locomotives would fly around without producing motion, even after the reverse had been demonstrated by experiment. In 1814, George Stephenson made his first locomotive with smooth wheels, which drew eight loaded wagons up a grade of twelve feet per mile; in 1815 he invented the steam blast, which doubled the power of his engine; and in 1816 he succeeded in producing an engine with simple and direct communication between the cylinders and the wheels, joint adhesion of all the wheels by horizontal connecting rods, and combustion excited by the exhaust steam. In 1818 we find him experimenting upon friction, and, in his own practical way, working out the problems upon which the success of the railway system depended.

The Stockton and Darlington Railroad, which was opened for traffic in 1825, was the most extensive railway enterprise thus far undertaken; it was twenty-five miles in length and had four inclined planes from half a mile to one mile in length, with intermediate grades of as much as fifty-one feet per mile. We have a discussion as to the gauge of this road, between the engineer, George Stephenson, and the proprietors, in 1824, from which it seems the question was settled with reference to the probable interchange of rolling stock, and the gauge of existing roads, which was four feet eight and a half inches; the same gauge was afterwards adopted on the Liverpool and Manchester road. Previous to the opening of this road, Stephenson established a shop for the manufacture of locomotives at Newcastle, and it is worthy of remark that the capital he put in this establishment was a testimonial he had received from the coal mining interests of the vicinity for the invention of the safety lamp, which has been so generally accredited to Sir Humphrey Davy.

At that shop were built the locomotives which, at the opening of the Stockton and Darlington Railroad, so astonished the world by drawing thirty-eight wagons, loaded with coal, merchandise and passengers, at the rate of twelve miles an hour.

The railroad excitement was now fairly ablaze; the canal and turn-

pike interests were alarmed; the land owners were startled at the prospect of inroads, which capital stood ready to back. An advocate proclaims that railroads "would widen the circle of intercourse, form a new creation, and extend the limits of industry and joy," whilst an opponent asks, "what is to become of the turnpikes, the coach and harness makers, and the horse breeders; the smoke and noise and hiss and whirl would dismay the cattle grazing in the meadows or ploughing in the fields. They would be the most complete disturbers of the quiet and comfort of the kingdom that the ingenuity of man had invented." The contest waxed warm in Parliament over the incorporation of the Liverpool and Manchester Railroad, but warmer in the field between the surveyors and land owners; the farm hands were sent out with pitchforks to drive off the engineers, who were stoned and their instruments broken. But we must leave George Stephenson and his friends with the difficulties they were so fully able to meet, and turn to America.

The railroad movement in the United States seems to have been coincident with the opening of the Stockton and Darlington Railroad. In 1827 we have an account of the opening of the Quincy Railroad in Massachusetts, extending from a granite quarry three miles to tide water, with a gauge of five feet; and the Mauch Chunk road, in Pennsylvania, from the Lehigh coal mines, nine miles to the canal, with a gauge of three and a half feet. The gauge of the former was probably induced by the wide and high wagon wheels used for moving heavy blocks of stone, whilst the latter is the gauge of the mining cars yet in use in that region. The Mauch Chunk road was built for a gravity road, and it has been so worked up to this time; mules were first used to haul back the empty cars, but stationary engines and inclined planes were afterwards substituted. We find the same gravity system, with nearly the same minimum grades, at work on some of the sections of the Stockton and Darlington Railroad two years earlier.

In 1827 we have the proceedings of a public meeting in Baltimore to forward the construction of a railroad thence to the Ohio River, and in 1828 the report of a Board, consisting of Col. S. H. Long, Capt. Wm. G. McNeill, and Dr. Wm. Howard, of the United States Engineers, appointed to reconnoitre the country to be traversed by it. Their report is elaborate and able, and is probably the most interesting document of its day on the subject of railways in America. It adopted Tredgold's recommendation of a four and a half feet gauge,

and defined with great clearness the topography of the country; but a remarkable feature in it is a want of appreciation of the forces to which railways would be subjected. The Board was afterwards commissioned to visit England to examine the railway system there.

About this time the invention of the Winans car seems to have made some diversion of the railway interests in its favor; it reduced friction one half, and on account of the great play provided for the axles they would adjust themselves to the radii of the curves. We have the most florid accounts of what a horse would draw in them, and curves were regarded as of small importance. It was thought these cars would do best with the flanges on the outside of the rails; hence the first track laid on the Baltimore and Ohio Railroad was of four and a half foot gauge for flanges outside, with very sharp curvatures.

But the railroad world was to be startled with another great event; the appearance, with a tubular boiler, of the "Rocket" on the rails of the Liverpool and Manchester Railroad in 1829, and the opening of that road for traffic in 1830. These events seem to have taken the breath out of our American railway magnates; the Commission to England did not report, but we find in 1830 a highly interesting and elaborate report from J. Knight on the subject of gauge, size of wheels, turning curves, &c.

Two miles of track had been laid on the Baltimore and Ohio Railroad for flanges outside, and some experimental cars had been run upon it, when an alarm note was sounded from England; a mistake had been made, the Winans car would run with flanges inside, and a model car was sent over. But doubts had already arisen as to the safety on any track, and this was the condition of affairs which brought Mr. Knight's strong mathematical mind to bear upon the subject, and produced his celebrated demonstration that the exact gauge for a railroad was four feet nine and a quarter inches.

It was the fashion of that day among engineers to criticise George Stephenson as a man of no science, but if a "principle in science is a rule in art," then was he a man of science, for he created, without much aid from books, the most important rules of art which have governed railroad operations up to the present day.

Somewhat in this vein was Mr. Knight. He had met no one in England who could inform him on what principle the curves were

turned; but he had observed that the wheels were conical, and he invented the "cone and cylinder" wheel—that is, a wheel with a flat tread, and a cone between the tread and the flange. The elements he had to deal with were, the radius of his curve, the diameter of his wheels, the play necessary for his cones between the tread and the flange, and all must be so arranged that an English locomotive of four feet eight and a half inch gauge could be used if necessary. He published his elaborate calculations, which resulted in a thirty-inch wheel and a four feet nine and a quarter inch gauge.

In 1835 Pambour visited the Liverpool and Manchester Railroad, and referring to the question of the cone as the means of turning curves says, "Thus far those means have been employed by approximation," and then, with a flourish of trumpets, gives the same rule for calculating that is found in Mr. Knight's report of 1830.

Gauge seems to have been chiefly a matter of local adaptation. Tredgold, writing in 1825, recommends four and a half feet for heavy freight and six feet for passengers; his argument rests entirely upon the necessity of keeping the center of gravity of the cars safely within the rails. Knight works up to four feet nine and a quarter inches in 1830 to suit his cones and short curves, and probably the gauge of four feet ten inches has resulted from an indisposition to deal with quarter inches. The compromise gauge which we are coming to between our four feet eight and a half inches and four feet ten inches "will be four feet nine and a quarter inches," a less mathematical mode of reaching the result than Mr. Knight employed. We have next the excitement over the wide gauge on the grounds of higher speed and more room, and now over the narrow gauge on the basis of economy. The gauges of this country hereafter will probably be four feet nine and a quarter inches and three feet, to be selected from local requirements; and the difference in the cost of building, equipping and working, and in the limits of capacity, will be nearly in proportion to the gauge.

In 1829 the "Rocket" made her debut in England with her tubular boiler; in 1830 the Liverpool and Manchester Railroad was opened, and the same year the first American engine appeared on American rails, built by Peter Cooper, of New York; it had a single cylinder of three and a half inches diameter, and run from Baltimore to Ellicott's Mills, a distance of thirteen miles, with a car and twenty-three passengers, at a rate of from five to eighteen miles per hour.

This was soon followed by a working model from P. Davis, a watchmaker, of York, Penn., which was afterwards expanded into the well known crab engine of the Baltimore and Ohio Railroad.

We meet here to-day upon the platform of mechanical progress. Forty-two years have elapsed, a period within the professional recollection of many here present, since the "Rocket," the creation of an uneducated Northumberland miner, blazoned forth, bearing upon her escutcheon all the essential elements of a modern locomotive. There are now upon the surface of this globe of ours 125,000 miles of railroads, enough to girt the earth five times, and there are plying upon them locomotives enough, if placed in a continuous line, to reach from New York to Chicago.

Mr. T. C. Clarke: I have always understood that our President, Mr. Horatio Allen, was the first person who operated a locomotive on a railroad in America. Can you give us any information on this subject?

The Chairman (Mr. E. S. Chesbrough): A friend of mine has said that Mr. Detmold claimed to have been the first man to operate a locomotive in this country, on a railroad in South Carolina.* I am, of course, unable to say whether it is so or not.

Mr. W. J. McAlpine: I do not intend to occupy the time of the Convention. I have a manuscript letter in regard to the first engineer on the Charleston and Hamburgh Railway, which, with many other papers, I will deposit in the archives of the Society.

Our most worthy President, Mr. Horatio Allen, was the first man who took hold of a lever to run a locomotive in America; and I, a boy at the time, saw it put together: it was called the Lion. Within the past ten years, a son of an old friend of mine, who is not present, has seen parts of it, as I have seen. Mr. John B. Jervis, on the Baltimore and Ohio Railroad, had two locomotives built, one by George Stephenson at Newcastle-on-Tyne, called the John Bull, and another by Governeur Kemble, at West Point, called the Brother Jonathan. It so happened that Mr. Asa Whitney, the master mechanic of the railroad at that time, did not quite know how to put these together. I, having served in my father's shop, knew a little about it, and thus helped to set up the first English locomotive.

Any person visiting London should by all means go to Kensington

* The South Carolina Railroad, from Charleston to Hamburgh, probably.

Museum, and there he will see the Rocket, the original English locomotive, a crude, curious thing—part of wood and part of metal, with an upright cylinder. He will also see one of the first steam engines made by James Watt, a stationary engine with walking beam and connecting rods of wood, and a most remarkable cylinder. A similar cylinder is in the Bank of England.

Mr. M. N. Forney: Some of the members of the Society have called attention to the present double truck locomotive. In 1832 Mr. Horatio Allen had two locomotives built for the South Carolina railroad, with double trucks, and a cylinder for each truck, which are the essential features ot the Fairlie engine. A lithograph of this locomotive is contained in the volume of printed testimony taken in the patent case of Winans eight wheel car, was copied by a paper in this country, and afterwards in London Engineering; since then Mr. Fairlie has taken out new patents covering the invention. I mention this as an instance of an invention once brought out and then abandoned, which is revived at this time to make the stir the Fairlie engine now does.

Mr. W. J. McAlpine: Both are entitled to the honor of the invention.

Mr. J. Dutton Steele: In looking over Tredgold's work of 1835 I may have been mistaken in respect to the date of the first engine. I recollect the opening of a portion of the South Carolina Railway, and of the thirteen miles of railway on which the Cooper engine was used nearly at the same time.

Mr. Collingwood: Will Mr. Theodore Allen state the reminiscences he may have heard about these engines?

Mr. Theodore Allen: All I remember is what I heard in conversation. So far as I recollect, Mr. Horatio Allen was sent to lay out a road, which, I think, was connected with a canal, (the Delaware and Hudson Canal) for the purpose of carrying coal; and the Board of Directors requested him to get any improvements by which they could carry coal by rail with steam power. He went over to England, purchased a locomotive, and brought it over here, arriving, I think, in June, 1829. The locomotive was put together in New York; and in order to see that none of the parts were missing, it was taken apart and again set up in July and August following. On August 7th, 1829, a number of persons were invited to see it start upon the road. A short distance from the station the rails were laid over a bridge or trestle work, built of unseasoned timber, which sprung considerably.

I have heard Mr. Allen say that he was not at all confident this structure would bear the locomotive, and so he determined to go over it alone. Placing his hand upon the throttle valve lever he started the engine, and moved off amid the cheers of the spectators. As he approached the bridge he thought whether it was the more prudent to go slow or fast; instantly he decided that if it were to fall it would do so at whatever speed he went, and he might as well go down handsomely, so he put on steam and disappeared from the view of the spectators. That was the first trip of a steam locomotive in this country. I am informed that afterwards he went to South Carolina and laid out a railroad, I think the first one there, with quite a length of roadway. The two locomotives first used on the road were built at the West Point Foundry; I have forgotten the names of these engines. A plan (which caused quite an excitement) for driving cars over the road by sails was tried there; with a single mast and a square sail, pretty good time was made—some twelve miles an hour, I believe.

The Chairman: The subject is exceedingly interesting. I have no doubt the older members could talk for hours upon it; but time is passing away, and other papers are to be presented. If there is no objection made by the Convention, I will call for the reading of another paper.

XXXIX.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

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AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

ELECTRO SCIENCE, AS A PART OF THE EDUCATION OF CIVIL ENGINEERS.

A paper read by STEPHEN CHESTER, Civil Engineer, Member of the Society.

There are few subjects with which, in a general way, the mass of the people are better acquainted than that of electricity.

Nearly every schoolboy passes through a period when his whole soul is absorbed in the preparation of glass bottles and Leydon jars; often maternal patience is tried to its utmost extent in the discovery of dilapidated tablecloths and varied colored garments, sacrificed in the interest of juvenile research in the school of Voltaic science.

Merchants, manufacturers and brokers are becoming more than amateur telegraphers, and in every department of industry, electricity is being used as a practical agent, and practical results are obtained. Yet there is no science of which so little is understood by practically scientific men.

I propose, therefore, to submit for consideration some of the reasons which seem to me to account for the apathy of that class of men in making exhaustive research, or even to exhibit general interest in an element already largely utilized by application in almost every known art, and assuredly destined at an early day to hold a position of still greater magnitude and importance.

For many years, known only in the phenomena exhibited in the chambers of natural philosophy and of educational institutions, the subject of electricity was regarded not as one of special or practical utility, but simply of such slight interest as might belong to the contemplation of any natural phenomena, and one of which no man of education and culture should be entirely ignorant.

Its first application to an important and useful art was to that of telegraphy, and so limited then in this field were the resources of scientific men, that the application of electro-magnetism to the purposes of distinct intercommunication was, and still continues to be regarded as an entirely new and original discovery.

We observe here that the application of electricity in this instance was not to accomplish more than a special and limited result, through the special and limited means of repeated electro-magnetic impulses. The problem to be solved was simply the production of controllable and recognizable signals at a distance. This accomplished, questions of economy, and expedient means of producing the element upon which these results depended were for after consideration, and were naturally esteemed of relatively insignificant importance.

The rapidly increasing demand for quick communication between distant places soon made electro telegraphy one of the most important business enterprises of the country, and under this stimulus lines sprang up in all parts of the United States and of the world; armies of employees soon became trained and skillful experts in the practice, and indeed in the general knowledge of the subject, so far as producing the special and limited results required in this specific application.

Hence, specific experimental knowledge of electricity as an applied science, has been almost exclusively confined to a class of men, who however bright and intelligent they may have been, were nevertheless entirely unqualified to develop largely any important and valuable result from their relatively wide experience, because:—first, they came almost wholly from a class of society, early thrown into active employment with but an ordinary school education:—second, no antecedents of employment or training had prepared them in capacity or disposition for the habits of exhaustive analysis required to develop a new science: and lastly, their field of observation and experiment has always been limited to the accomplishment of a specific result.

In later years, the many new and useful applications of electricity

in arts, other than electro telegraphy, has stimulated the competitive efforts of manufacturers to further and more important applications. Under this stimulus, the attention of the scientific world has of late been drawn more directly to the subject, and men like Seimm, Clark, Becquerel, Sabine, Dumoncel and others, have made valuable additions to general science in elaborate works upon electricity.

But these are for the most part "men of the schools," and while their books evince pat ent study and careful analysis, the character of our standard works is not such as we would expect and require from men, who from previous training and long habits of practically applying abstruse theorems to actual construction and problems to be solved, would naturally investigate this subject rather in its application to specific results, than in its general connection with abstract science.

Again growing out of the antecedents that I have mentioned, is a popular and widely different error, that this is not a positive science, (if I may use the expression in this connection); that all knowledge of the laws governing this subject must of necessity be vague—uncertain, and beyond the limits of mathematical measurement and calculation; and that this mysterious invisible element is recognizable only in, and through, the phenomena attending its vibrations and movements, and by which then, the experienced expert may judge of the presence of the unseen agent, with about as great a degree of certainty as the physician forms his diagnosis of disease—from the varying symptoms of his patient.

On the contrary, notwithstanding the fact that our researches in this direction have been hitherto desultory, and far from exhaustive in their nature, and our acquaintance with the laws governing the operations of electricity are very limited,—yet we have developed this fact with great certainty, that it is no uncertain science, but one susceptible of exact analysis conforming to arbitrary, positive rules and calculation. This invisible, silent and mysterious force can be measured in its length, breadth, and thickness, with the accuracy that the beam of a truss may be measured: the consumpt on of material required to develop it in known and determinable qualities, may be as closely estimated as the effective working amount of steam can be computed, from the amount of coal burned: and the electro motive effect of this force under any given conditions, may be as exactly calculated as that of the thrust of a brace under given conditions. I do

not say that this has also been literally accomplished; but such advances have been made, that the ultimate result is no longer a matter even of probability.

Already European governments have made this subject one of the principal studies in the course of military education. In this country, our military and naval schools have included in the courses, with instructions of a general character, the application of electricity to specific, offensive, and defensive objects, and we have both in the army and navy, schools for teaching in torpedo and signal service. It is not necessary, if the time would now permit, to point out, how inadequate such instruction must be, except so far as to accomplish the specific limited object in view.

To the civil engineers of the country, men peculiarly fitted to the task by reason of special training and experience, *mechanics* rather than chemists, philosophers and abstract scientists, belong the duty of developing this subject to the dignity of a practical science.

As my time is nearly exhausted, let me add one reason why this subject should be entrusted to the hands of practical engineers, and not only to philosophers and students of abstruse science—*the public welfare demands it*.

Under cover of our general American apathy, to which I have before alluded, the popular impression has been largely cultivated, that this is a vague, uncertain, eccentric power, its laws unknown and undiscoverable, and that, the simple manipulations of telegraphy could be acquired only by special experts ; and more by reason of this popular ignorance, than from any other cause, extensive telegraphic monopolies have been built up to which we are paying a profit of 400 per cent. for every message we send by them.

Lastly, let me remark to the body of men before me who have attained individually to reputations of which the country may well be proud, and collectively to an eminence second to that of no similar body in the world, I cannot but think it a burning shame, that the greatest, richest, and politically the most powerful telegraph company in America, if not in the world, should, from necessity, if not from a less reputable reason have been induced, to secure the services of an English expert to instruct operators in the first principles of their own especial calling.

It is hardly to be doubted for a moment that this science, entering as it already does, and will through so many devices, into the manipula-

tions of many useful arts, will, in the future, form an indispensable part of the education of civil engineers. In our own time and generation we may anticipate this inevitable result.

Mr. COLLINGWOOD: Will Mr. Chester state some of the important effects of electricity. It is used, I am sure, in the arts, beyond what many of us know.

Mr. CHESTER: I presume it would be a very interesting subject, if time permitted, but it is one that requires some degree of preparation to present to the Society in an instructive form.

I will say that generally, it is used with different devices for controlling automatic machinery—controlling it in the sense that the brain and nerves may control the muscular system; not as a positive force, but as that which governs a positive force. There is hardly a limit to the number of applications which might be devised, for applying electricity in this way, in the several branches of manufacturing art.

Mr. CORYELL: Cannot electricity or galvanism be brought to bear in testing the strength of iron and steel? Will not the arrangement of the light and the position of the rays assist in determining the quality of iron and steel?

Mr. CHESTER: Experiments of this kind have been made. The capacity for conducting electricity is an almost positive test of the purity of copper. The same test, or a test of a similar nature may be applied with very exact results to iron. Experiments have been made, I understand (I do not know how far they have been carried beyond the limit of my own experience), in testing the quality of iron by magnetic impulses.

XI.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

ENGINEERING.

A paper presented by W. MILNOR ROBERTS, Civil Engineer, Member of the Society.

A history of engineering would fill many books. A list even, of engineering constructions worthy of special study would fill a volume. The mere reference to the names and characters of those who have passed their lives in engineering pursuits would require more space than can be assigned to a mere paper upon the general subject. The theme, in its nature, opens an illimitable field for consideration and reflection. How then, within the compass of a few pages, can an article be shaped which shall possess sufficient attractiveness to engage the attention and sympathetic interest of the eminently practical men who meet together in this Society for mutual improvement?

Mutual improvement is the key-note, watchword, and cardinal, fundamental principle and constitution of our American Society of Civil Engineers; and accessory thereto is the encouragement of mutual sympathy. Any effort, therefore, however crude, which shall tend to elevate the profession, or to present its prominent features in a pleasing aspect, may, it is hoped, not be wholly unacceptable.

It is not designed however to offer specially a eulogy of engineering, or of engineers; but rather to glance at such salient points as may

occur to the mind, hoping for, rather than promising satisfactory results.

In thought, we can quickly recur to a period in the early age of man when there was neither engineering nor engineers; when even agriculture, the primary field of human labor, had not yet originated; when the only employment was the simple task of tending flocks and herds—which did not necessarily bring into requisition engineering skill. Nor is it presumable, that in the first rude steps of agriculture, in the rudimentary work of sowing and planting, any special engineering talent was needed.

The early and primary engineering thought may have been, that which discovered or invented the most primitive implement of husbandry; even were it no more than the dragging of tree limbs to loosen the surface of the earth; the object being, to assist nature, and to extend the area of human labor; the two chief aims of engineering efforts.

Providence, the sublime architect and engineer of the universe, has made life itself a thing of labor, and has endowed our race with the requisite facilities to direct it intelligently. Yet man was not permitted to step full-fledged upon the earth, possessed of a perfect knowledge of his Creator; but with a very little, only suited to the needs of an infant but progressive being; to be increased continually from the exhaustless fountain whence knowledge perennially flows, always in exact accordance with the divine plan.

As population incr ased, and formed society, and as the wants of mankind multiplied, new inventions were sought; not by every individual but rather by comparatively few, the early engineering minds. Not that they were then called or known as engineers, or that science of any kind had yet taken root in the world; nevertheless, there were the engineering minds of those ancient days, corresponding, though under different circumstances, with the engineering minds of modern times.

Gradually, in the order of events ordained by Supreme Wisdom, there sprang up the necessity for, and with it the actual division of labor whereby men were separated into different classes. Very early, in a progressive world, there must have come the makers of implements as a body distinct from those who used them; although then, as now, the two may frequently have been blended. Diversity in human occupation is clearly an imperative law of a world of human

beings, such as ours; and as the numbers and wants of mankind increase, the division of labor naturally becomes more and more minute.

The labors of the engineer, in all ages, beginning almost with the earliest times, have been devoted specially to the attainment of practical results, tending to benefit mankind by the introduction of improved methods of performing the ever augmenting and ever changing work of the world. This is true, whether considered with reference to the humblest new implement or instrument, or to greater works, the planning or construction of which may have been, to some extent, dependent upon previous engineering inventions.

Let us here draw a friendly line of demarkation between engineering and engineers on the one hand, and art and artists on the other. Not that artists may not be engineers and engineers artists, for such is not an uncommon union; yet there remains a real distinction between the two professions, founded upon that basic law resulting in both division of labor and diversity of pursuits.

In the general view here attempted, it might be granted that artists, as well as all inventors of mechanism, of whatever nature, could range on the same general plane. Numerous inventions, and a greatly extended interchange of products, the proceeds of agricultural and mechanical operations, must have arisen before there could have been the necessity for an engineer, or of a special engineering profession. On so nice a point, the ancient histories are too vague to enable us to define accurately the advent of engineering proper. There are, however, many noble structures, of which only the ruins now exist, showing that ages ago, there were engineering minds devoted to devising and executing great works, demanded in those days either for practical use or ambitious ornament. Engineering as a science may not then have had existence; yet, in the nature of the case we may fairly assume that engineering minds alone could have conceived, arranged and executed those early proofs of extraordinary human genius. Who can say how many triumphs of engineering skill, once, perhaps, the admiration of the world, may have disappeared without having any living historical record?

We find enough extant to demonstrate that the human mind has almost or quite from the beginning, been constituted with capabilities enabling men to cope successfully with natural obstacles, which, doubtless for wise purposes, have been such as to call forth the exercise of their invention.

At some long past period in the world's history, architecture must have assumed the most imposing dimensions, judging merely from the glorious architectural monuments of genius which have been preserved and which have excited the wonder and admiration of succeeding ages. Architects, whether designated by that technical title or some other, were in truth the great engineers of their day and generation. The labor of the architect, whether ancient or modern, whether relating to the useful or to the ornamental, in their myriad combinations, are those which we here ascribe to, and include under the generic title of engineering ; although in our modern days, owing to the vast increase in the number and variety of human wants and luxuries, the inevitable consequence of advancing civilization; the professions of architect and engineer have become distinct and separate, a convenient and in many respects a necessary division of labor.

Michael Angelo, the architect of St. Peter's, was an engineer of the very highest order, although he may not at any time have been called upon to plan or execute engineering works proper, as we now entitle them. In the designing, planning and execution of that masterpiece of human thought, the grandest exercise of true engineering science and skill must have been brought into requisition.

It is not the particular duty, or even the particular direction in which engineering skill may be exhibited, which can define the real character and power of the engineer ; it might easily happen that an engineer's whole life would pass in a manner never to call forth the perfect manifestation of his peculiar internal engineering strength :

"Full many a rose is born to blush unseen."

When studying, for example, such a work as St. Peter's, who can resist the conviction that he is contemplating a magnificent emanation from a master engineering mind ? Could it be written ; could the history of the entire working of that mind be laid bare, from the moment of its inception of the grand edifice to the last fading thought of the dying architect. it would be more intensely interesting than the most thrilling romance that the most lively fancy has ever conceived. Imagination, invention, skillful combination, judgment, all of these, and the greatest of these, judgment, must have had full exercise through years of toil of the brain of the great architect.

If required to designate the one essential requisite of an engineer, we would place judgment first. Even as "action, action, action," is

to the orator, so is judgment, judgment, judgment, to the engineer. An engineer to be great, or strong, must have other qualifications ; but, lacking that essential element, judgment, he would be incapable of availing himself advantageously of the practical exercise even of brilliant talents.

All men, including engineers, are, of course, subject to errors of judgment ; the nature and extent of such errors, in an engineer, must necessarily determine his status, as well as his usefulness. Many minor errors, if any there were, in the engineering of such a structure as St. Peter's, might be overlooked or very leniently regarded in view of the magnificent result.

In every age the engineering mind has naturally been turned to the planning and executing of works fitted to the time. The character of even modern engineering has consequently assumed different aspects at various periods. The engineering which resulted in the successful floating of Noah's ark was peculiar to that age and occasion, and had relation to the object for which it was designed, and which it answered. The engineering which culminated in the launching of the Great Eastern safely, on the bosom of the ocean, was of a different type; only rendered possible because of a vast chain of inventions and engineering constructions, the accumulation, as it were, of ages of experience. Had Brunel, the distinguished engineer of the Great Eastern, lived at the time of the deluge, it is not probable that he would have designed his stupendous naval triumph, for there were thousands of inventions and works to intervene ere the practical conception of the Great Eastern could become possible, even to the greatest engineering intellect. The world's engineering mind had to grow up to it, through experience. We do not ascribe the conception and completion of the Great Eastern in any degree to accident ; it may be regarded rather as the natural sequence of preceding engineering knowledge, combined with other favoring circumstances, among which was the vast accumulation of commercial capital in Great Britain.

Few believed that a vessel so large, so costly in construction, and so expensive to use, would prove to be a financial success ; nevertheless the great design of the bold engineer was successfully consummated, although it is considered that by overtasking his brain, Brunel sacrificed his life.

For many generations in comparatively modern times, canals, and their various appliances, constituted the principal engineering work of

the world, and chiefly in connection with navigation, as a means of facilitating transportation. Although tram roads were used many centuries ago, the railroad of our day is undoubtedly a modern invention; coming when the world needed it, and not before. It is the most effective engineering machine in connection with transportation that has ever appeared in the world, and is now the strongest lever in extending the area of civilized settlements over vast tracts, which but for its wonderful facilities might lie waste for ages more. What engineer witnessing the insignificant beginning in Pennsylvania and Massachusetts forty-five years ago, when the first regular railroad in the United States was inaugurated for the transportation of passengers and freight, between Mauch Chunk and the Broad Top coal mines, nine miles,—and the Quincy Granite Quarry road, four miles long,—would have been bold enough to predict the present condition of railroad engineering in the United States, and in the world!

The entire railroad system throughout the world, which now so largely controls its commercial and other movements, in war as well as in peace, has germinated and grown to its present gigantic proportions within less than half a century. The locomotive, and the ocean steamer has been vouchsafed to mankind within the same brief period. Gradually, from doubtful and insignificant beginnings, railroads, the triumph of modern engineering skill, have extended the domain of their influence, till now they span continents, and unite in iron bonds the commercial and social interests of Europe, of the States and Territories of our own country as well as extensive regions in British North America, and the principal countries of South America; and they are rapidly extending into some of the regions of Asia. In fact, the world's great interchange of travel and commerce, as well as the beautiful arts of peace, and, sad to say, the terrible paraphernalia of battling nations, are now regulated through the medium of the railroad and the steamship.

Hand in hand with engineering advancement in this direction, as exhibited in almost countless mechanical inventions and improvements now connected directly and indirectly with railroads, science has lent its powerful aid in varied forms, all acting in harmony with the grand movement of the age. Water, fire, lightning, and even the atmosphere, have been put in harness by the engineer, and made to perform herculean

labors with perfect regularity, certainty, and reliability, and with a vast saving of expense and human labor.

Without the telegraph, the railroad would be a far less valuable machine. Nothing has added so much, in so short a time, to the practical value of the railroad as the telegraph. All honor to its distinguished inventor, the late venerable Professor Morse, whom we have recently followed to his last earthly home, full of years, respected and honored throughout the civilized world. What the nerve force is to the human frame, the telegraph is to the railroad system, enduing it with almost intellectual vitality, and controlling and guiding its every movement.

Quite recently, within a very few years, the engineering mind has designed and perfected the employment of compressed air, not only as a force in penetrating vast mountain barriers, but as a power employed in descending to the depths of rivers and harbors, and founding securely upon the solid bottom, gigantic piers of massive masonry. This has been accomplished at the St. Louis bridge, even to the great depth of one hundred and ten feet below the surface of the water, in a rapidly flowing river.* Thus as obstacles present themselves, either in the way of the location, the construction, or the working of railroads, modern engineering skill meets and overcomes them.

Although during the last third of a century railroads have chiefly occupied the engineering talent of the world, canals, rivers and lakes still exist, and in their construction or improvement, as the case may be, give rise to important engineering problems and works. Previous to the introduction of the railroad, the principal work, especially of the American engineer, was connected in some way with hydraulic operations, whereas now these are quite secondary. One cause of the change, particularly in Europe, may be traced to the fact that most of the principal practical water lines have already been improved. Even in the United States, when yet in comparative infancy, engineers had designed and completed a great number of grand water communications, including the Erie Canal, three hundred and sixty miles long, uniting the extensive chain of our inland lakes with the Atlantic Ocean; this, and the canal systems of Pennsylvania, Maryland, Virginia, Ohio, Indiana and Illinois, are monuments to the hydraulic

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* And also at the East River Bridge, to the depth of eighty feet.

engineering talent in our country during the era which immediately preceded the railroad period. Canadian engineers were not behind in this branch of engineering; a magnificent line of water communication, largely artificial, connecting the lakes with the ocean, by way of the St. Lawrence river, attest to both their skill and energy.

Forty years ago, the age of canal engineering culminated in the United States, and most of the engineers who were distinguished in that line now rest from their labors. A few transferred the field of their professional duties from the canal to the railroad, but nearly all of them have left this earthly sphere to occupy their minds on another plane of existence.

Although, in many important particulars, the general routine of engineering duties on canals and on railroads may be similar, yet there are material points of difference; and this is becoming more and more marked, as year after year new triumphs appear in the vast system of railroad engineering; whilst in connection with canal operations, owing to various causes, little change is observable.

In general, the profession of engineering seems to present peculiar attractions to the youth of our country, partly owing to the roving nature very often of its service, and partly because, of late years, it has afforded opportunities of connection with important public enterprises, and occasionally conferred wealth upon the speculative engineer. Some, conspicuous among engineers, have turned their attention chiefly to the manipulation of the financial aspects of railroads, and have not devoted their best energies to the scientific and practical departments of engineering proper.

Most engineers, however, in our country, have been true laborers in a laborious profession. As a rule, the working engineer in the United States might range himself in the company of the great Doctor Johnson, who said. "It is the fate of those who toil in the lower employments of life, to be rather driven by the fear of evil than attracted by the prospect of good; to be exposed to censure without hope of praise."

Nothing among professional employments can be more precarious than the active life of an engineer; and generally the remuneration for services performed is quite inadequate, compared with the degree of toil and brain work required of the majority of working civil engineers in the United States. Yet the engineer is usually made the arbiter and final umpire between the parties owning the greatest

enterprises, and those undertaking the construction of their works. He has to survey, plan, and locate; instruct, direct, and judge; respecting almost every conceivable aspect of numberless questions which arise in the conduct of every considerable railroad or other public enterprise; and, with rare exceptions, this delicate and responsible duty has been faithfully and impartially performed. In every profession there may be found mercenary individuals; yet it may be remarked that among civil engineers there is a certain high-toned feeling, an *esprit de corps*, which tends to guard the engineer against a temptation to falsify or deceive. The honorable engineer, if he has also the requisite ability and practical experience, is justly regarded as a valuable member of society. No profession involves a greater range of study, although in others the mind may be equally tasked in mastering the manifold points which appertain to the particular mental pursuit, whether it be law, medicine, or theology. The mechanical powers, the nature and strength of materials to be employed in construction, as well as of the various earths and rocks met with in traversing the country, the effect of different gradients and curves, the proper adaptation of means to the end proposed; geography, topography, and climatic influences; the period and action of floods, and of droughts; forces required to accomplish given results in stated times; economy, with judicious appropriateness in planning structures; readiness and accuracy in calculation; these, and many other things, must be familiar to the mind of the accomplished engineer.

The day has not arrived in the United States, and perhaps it may yet be distant, when an American Society of Civil Engineers shall resemble very closely the London Institution, for the reason, chiefly, that the vastly greater extent of the American field prevents concentration even in this already great metropolis, of a sufficient number of civil engineers, having time at their disposal, for the reunions which, even under our American disadvantages, are so beneficial in their social as well as professional aspects.

The majority of American engineers occupied with the manifold and engrossing practical duties which their engagements devolve upon them, can scarcely find time to keep up with the current engineering literature of the day. The number of engineers all over the world has become so considerable, that even moderate literary contributions from individuals, when aggregated, in a brief period constitute libraries. This is one of the modern phases of engineering life in the world.

Yet in this particular, engineering only partakes of the general progress; other professions experience analagous accumulations of contributions to the science of their respective occupations; but with the exception of the military, no other profession, like that of civil engineering, carries its votaries over such widely extended areas.

The military is, however, largely an engineering profession, although but one of its arms is devoted specially to the practice of engineering pursuits proper. Yet, in the admirable engineering school at West Point, all who become officers are trained to some extent in engineering studies. It is true that there are peculiar qualities necessary to the great soldier, which have but slight reference to engineering in its ordinary sense; yet, in many respects, the soldier will be the better as a soldier, if possessed of practical engineering talent. The superior military engineer must have elements which may easily be turned to valuable account in the quiet pursuits of civil engineering. The same steel which glitters aloft, waving men to battle, may be turned into the peaceful furrows made by the ploughshare of the civil engineer, designed to benefit, not to destroy.

Although the thorough knowledge of any science calls for the incessant attention of almost any single mind, yet, the intercommunication or homogeniety, so to speak, of all science, demands that every scientist must be more or less familiar with every branch; and every civil engineer is presumed to be also a scientist; in fact, in the present advanced and advancing age all science in some of its varied forms appears to be assimilated with the great onward movement of mankind both in knowledge and works.

The power of accumulative inventions, of invention founded upon invention, is daily manifesting itself in a wonderful succession, the final limits of which, losing themselves in the mystery of infinity, man can never attain; but in this pursuit of perfection, man derives all that is of real value in the way of a satisfying human happiness. That rest, and absolute contentment, which by some is attached to the idea of supreme happiness, it is obvious can only be secured by the sacrifice or oblivion of the most valuable of human characteristics—memory and reflection. True, if these could be replaced by something purely heavenly—some indescribable and different order of functions above and beyond the present range and scope even of human imagination, it is reasonable to hope that under the due administration of the laws of Providence we should be prepared to receive and appreciate it.

It may not be unphilosophical to indicate that we seem even now to be nearing a new order of scientific triumphs. The striking advances made during the last quarter of a century in the practical working of telegraphy, of itself furnishes strong confirmatory evidence in this direction. Keeping in mind the plain fact of the illimitable nature of invention, and noting the remarkable improvements which have already been effected in such a brief period, to extend and facilitate social, commercial and political intercourse, it can hardly be deemed unreasonable to infer that we are yet only on the verge of a vastly greater field, soon to be commanded by succeeding inventions all operating in harmony with, and aiding to extend still further the facilities which are demanded by the ever progressive and ever accumulative wants and practical necessities of the human race.

If there be in the universe an all pervading intelligence, as both reason and revelation assure us there is; and if all we know is to us but the tangible emanation from that infinite source, how can we assign limits to future progress in knowledge, and when will the supreme intelligence, if ever, cease to bestow upon mankind gifts of intellectual advancement?

It appears to be peculiarly the province of the engineer to adapt every new discovery to its possible new functions in the economy of the world. Nothing is too small, nothing is too great, to be employed in the application of knowledge to practical objects. Is it presumptuous, then, to anticipate that the day is approaching when the engineering wonders of the present age shall be eclipsed by others yet to be discovered? Either knowledge and invention must cease to be accumulative as they now are, or the progress of mind and its practical triumphs must go on for ever.

The greatest powers yet permitted by Providence to be controlled or extensively manipulated by human intelligence are invisible, and only known to us through their effects. Gravity, air, steam, magnetism, and nerve force cannot be seen or heard; their practical operations are nevertheless becoming daily more and more important in every branch of the world's engineering. When we reflect upon the wonderful changes which the knowledge and use of those powers have effected within a very brief period, the future, if we could see it, would probably be startling. How soon we become not merely reconciled to, but familiar with operations which, when yet undeveloped or not fully comprehended, excite only our wonder. The self-

sufficient may be inclined to say, "We have now arrived at or near the summit of human invention and progress; we understand all the forces of nature; we comprehend all the elements with which we must work; and for want of new elements or new forces there can be little room between the present attainments of science and an absolute limit." This is a very limited view, however, not warranted by our past experience; and it may be no more true to-day than a similar remark would have been true a century ago.

Therefore, with all due respect to our ancient predecessors, let us look forward to triumphs of engineering science in the future even greater than any which have yet marked the world's progress, and commensurate with the augmented and augmenting knowledge, which thus far seems to have no bounds assigned to it by our beneficent Creator.

XLI.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE,—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

AN AUTOMATIC OR GOVERNOR CUT-OFF FOR STEAM ENGINES.

A paper read by WILLIAM ARTHUR, Mechanical Engineer, Member of the Society.

The subject of this paper is, in the economy of working, one of the most important attachments to the steam engine; namely, an automatic or governor cut-off.

The advantages of expansion were early recognized by Watt, in fact they are comprehended in his patent of 1782. The self-acting, automatic or governor cut-off is of more recent introduction, and has been the subject of many patents and much litigation, yet I have not known any one to claim that the arrangement here described infringed on any other, and some cut-offs which have been considered important are *de facto* the same as this in principle, the only difference being an application at each end of the cylinder to save steam in the passages; when, at best, the loss is small, being only that steam which has expanded after having performed its work; and even this loss is compensated for by the decrease of radiation from high pressure steam, due to the use of a small steam chest.

This cut-off I have known favorably for thirty years. I have manu-

factured and used it in the United States for nearly twenty years, yet such is the desire for something new or patent that this, attractive as it should be to the scientific engineer, appears to me to have quite escaped his notice ; in like manner, many good things are allowed to pass away.

This cut-off has been manufactured at the Atlantic Steam Engine Works in Brooklyn, N. Y., for many years, and, with a wish to diffuse knowledge, I will now lay before this Society, for the inspection of engineers, a description of its details. The drawings presented represent the most recent arrangement of the cut-off, as now being applied to a horizontal steam engine of 100 horse power, erected by these works for Messrs. Colgate, at their soap factory in Jersey City, in 1863, and now being reconstructed for the first time.

Fig. 1 is a side elevation, Fig. 2 a plan, and Fig. 3 an end view, mostly in section. The same letters in each represent the same parts. The horizontal lever A, fixed in the rock shaft B, is attached at its forked extremity to the grooved collar of an ordinary governor, which, when raised or depressed, thereby operates the vertical lever C, (fixed on the same rock shaft); and by a connecting rod, the secondary vertical lever C C. This lever grasps the grooved collar of the cam D, and under the action of the governor, moves the cam back and forth on the square of the shaft E, which shaft (geared directly to the crank shaft), has a definite rotation. The vertical lever G, bearing at its forked extremity the friction roller F—in contact with the surface of cam D—is fixed on the rock shaft H ; the horizontal lever I is fixed on the same rock shaft, and is connected with the spindle of the cut-off valve J ; thus, as the governor oscillates, more or less steam is admitted ino the steam chest. By this arrangement the lifting projection of the cam being parallel with the axis of the driving shaft, whatever may be the longitudinal position of the cam, the valve will always be opened at the beginning of the stroke; while, as the cam is moved back or forth on the shaft, its tapered or wedged part, presents a larger or smaller surface to the friction roller, and the valve is closed later or earlier during the stroke.

The steam chest is an ordinary short chest, with a short valve covering the steam and exhaust ports of the engine ; here the slide valve, which is usually a steam and exhaust valve, becomes an exhaust valve alone, and the cut-off valve acts as a steam valve.

The gear wheel, on the longitudinal shaft E, is one half as large as

that on the crank shaft, thus the cut-off valve is lifted twice during every revolution of the engine or once at each stroke.

The cut-off valve is double faced and nearly balanced, the only difference in face, being that necessary to allow the lower face to slip past the upper seat : it is worked with great ease.

Thus is simply explained this attachment so far as applied to the ordinary horizontal high pressure steam engine; various modifications will suggest themselves to the practical engineer, according to the style of engine to which it is to be attached.

I am confident I have seen no cut-off more efficient, economical and simple than this; even its disarrangement causes no permanent stoppage of the engine; all that is required being to raise the cut-off valve, the exhaust valve will then act as an ordinary short slide valve, and allow the engine to work with unvariable steam until the cut-off can be put in order again.

It is obviously necessary that the steam chest should be small, therefore it is preferred to have what is strictly the exhaust valve chest—while the cut-off is operating—as small as possible ; the ordinary steam chest attached to many engines has a capacity nearly as large as the cylinder, in which case were this cut-off applied it would be entirely inoperative.

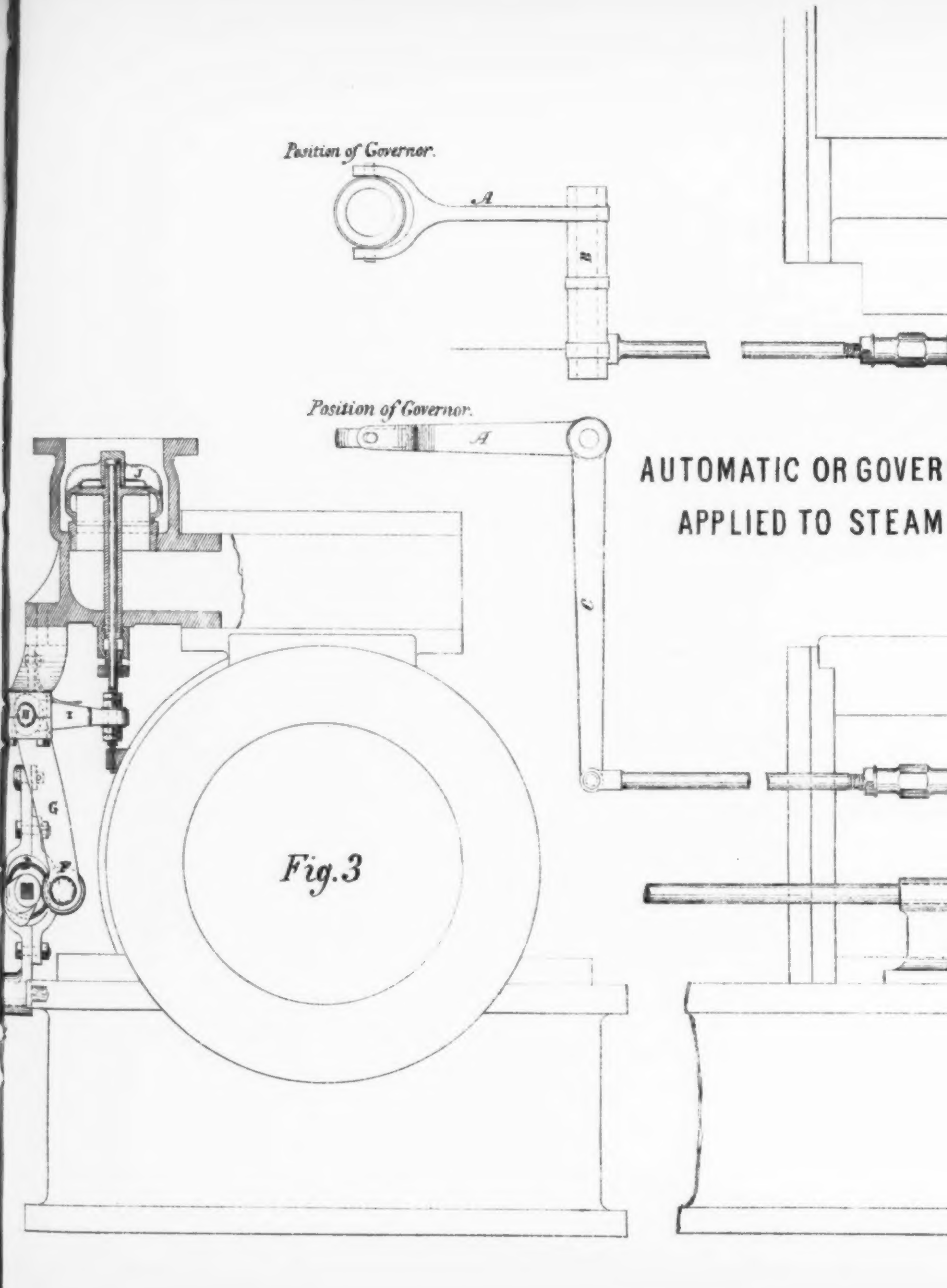
This cut-off I first attached to an engine made in 1854 for the Brooklyn Flint Glass Company, with fourteen inch cylinder and three and a half inch stroke; and since, among others, to a vertical engine driving Messrs. Leroy's Lead Works, Water street, New York, erected in 1859, with sixteen inch cylinder and four feet stroke; a horizontal engine driving Mr. Scofield's Planing and Moulding Mill, Twenty-seventh street and First avenue, New York, erected in 1860, with twenty inch cylinder and four feet stroke; a similar engine running Mr. Loomis's Planing Mill; the engine at Messrs. Colgate's Soap Factory, before mentioned, and to the one at the Atlantic Steam Engine Works, Brooklyn, New York, which has been in operation about eleven years.

The first time I saw this cut-off working must be about thirty-one years ago; it was on a condensing beam engine, manufactured by J. and A. Blythe, of Lime House, London, for the Dye Extract Company's Works, on the Isle of Dogs, opposite Deptford, England; and although the details were somewhat different from those here described, the principle was the same.

The earliest application of this cut-off I have seen noticed is

described in "Tredgold on the Steam Engine," (vol. 3, page 12 text, and plate xvi, last edition,) to what is there called Maudsley's engine; it is also noticed in "Civil Engineers and Architect's Journal," 1842, (vol. 5, page 252), as introduced by James Whitelaw, of Glasgow; and in "Cyclopædia of Machinery" (page 45 text), where it is described as a regular cam or projection, attached to the governor, and sliding on a spiral feather fixed in a shaft, thus moving so as to admit steam into the chest, at or before the time the engine is on its centres; the steam being prevented from entering the cylinder by the succeeding valve. It is also noticed in "Papers on the most advantagious Use of Steam," by Professor Gordon, read before the Glasgow Philosophical Society, Nov. 6, 1844, and published in "Practical Mechanic's and Engineer's Magazine," 1845, (vol. 4, page 138); wherein he states, this plan has been carried to perfection by Edwards, who attached a variable expansion gear to the governor, and by Sterling of Dundee; and in "Civil Engineer's and Architect's Journal," 1845, (vol. 8, page 57,) wherein Stoltze's engine, exhibited at the French Exposition of 1844, is shown with governor expansive valve gear. For further particulars see "Tredgold on the Steam Engine," (vol. 2, page 283, Division B,) where the invention, as applied to Maudsley's engine first spoken of, is ascribed to Joshua Field.

This device was first patented in the United States by W. McCammon of Albany, N. Y., May 22d, 1849, and afterwards by Waterman and Corliss, in 1851, and Wright in 1854.



Position of Governor.
A
B
Position of Governor.
A
C
J
H
I
G
F
AUTOMATIC OR GOVER
APPLIED TO STEAM

Fig. 3

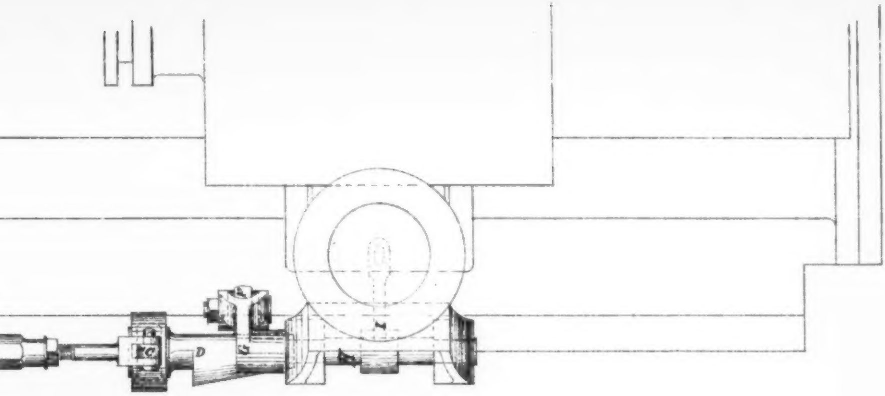


Fig. 2

VERNOR CUT OFF

AM ENGINES.

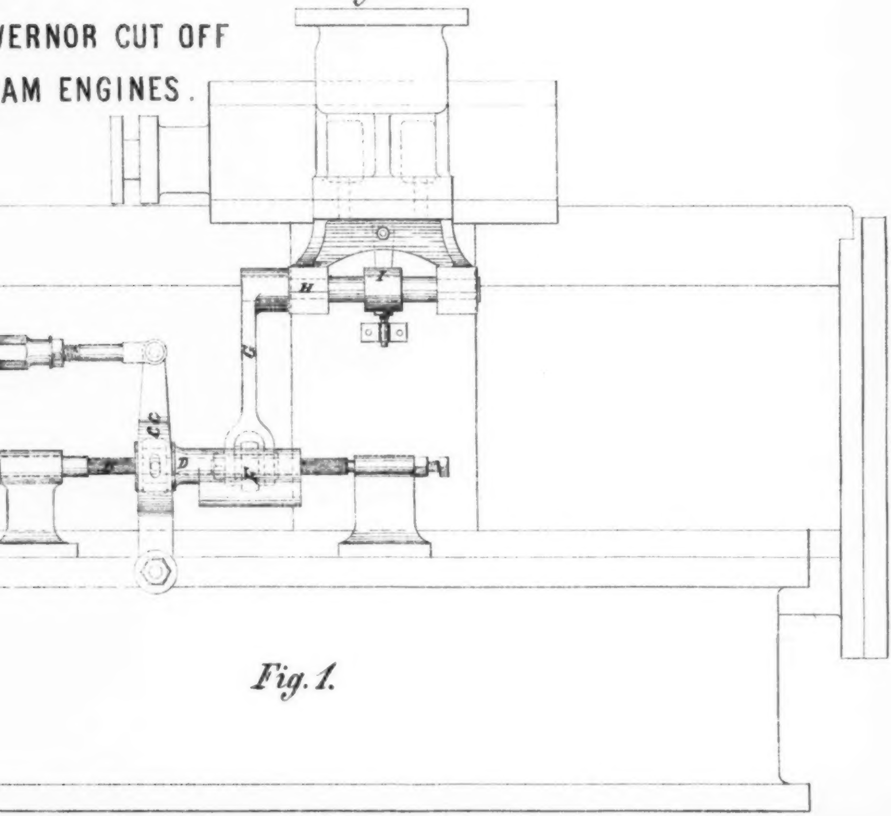


Fig. 1.

AM. PHOTO-LITHOGRAPHIC CO. N.Y. (OSBORNE'S PROCESS)

XLII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

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AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

SKETCH OF THE PLANS AND PROGRESS OF THE DETROIT RIVER TUNNEL.

A paper read by E. S. CHESBROUGH, Civil Engineer, Member of the Society.

The first suggestion of a tunnel under the Detroit River* for railroad purposes was made to me about five years since by Mr. James F. Joy, President of the Michigan Central Railroad, with the request that the feasibility of the project might be ascertained as soon as convenient. In January, 1869, a report, accompanied by plans and estimates was presented to that gentleman. From these are taken most of the statements in this paper, so far as they relate to what was done previous to the actual commencement of the work.

In the preliminary examinations the first attempt was made to get a line at right-angles with the river, not only across the stream itself but between the extremities of the approaches on each side. To avoid as far as possible damages to valuable property, the lower side of the city was selected. But boring showed very unfavorable ground there, and as the bed rock in that vicinity lies too near the surface for conven-

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*All who are familiar with the Detroit River know that at the particular point where the tunnel is located, it is about half a mile wide, and runs from west to east or very nearly so.

ience, and is still nearer lower down the stream, further operations were necessarily confined either within the limits of the city or above it.

Careful examinations were made above the line first surveyed, by borings on each side of the river. When good ground was found on each side at points opposite each other, borings were made in the river between ; but it was not until the vicinity of the Detroit and Milwaukee Railway Depot was reached, that ground considered safe entirely across the river, was met with.

A perfectly feasible line coul l be obtained entirely above the city, where the ground is very favorable, but in consequence of a greater length of tunnel and considerably longer new track being required, nothing could be saved in expense by it, and much lost in convenience ; for the line adopted makes the utmost possible use of existing depot grounds of the Michigan Central Railroad on the Detroit side, and will not require a great sacrifice of the station buildings and tracks of the Great Western Railway on the Canada side. These two companies are the parties most interested in the enterprise.

The line as finally determined upon commences at the Michigan Central Railroad passenger station, passes for a considerable distance along Jefferson avenue under the southwest corner of the Biddle House and the east end of the freight house of the Detroit and Milwaukee Railway, crosses to the other side of the river about half a mile above the terminus of the Great Western Railway and unites with that road about one and a half miles further on, making the entire length of track between the two roads a little less than three miles. The length of tunnel is to be about 8.600 feet (3,000 of which is under the river.) The least curvature on this portion of the line is 2,300 feet radius, and the steepest grade, one in fifty.

Although the borings indicated favorable ground along the line adopted, except that possibly there might be a very troublesome amount of water from land springs, or springs having their source higher than the river, it was deemed prudent to avoid all risks as far as possible in planning the details of the work. For this reason, it was determined to have not less than twenty feet of roof between the top of the masonry and the bottom of the river. In no case, as far as could be ascertained, would there be less than twelve feet of stiff clay immediately above the tunnel.

To diminish the risk still further, it was determined to make two

single track tunnels, circular in form, with eighteen and a half feet interior diameter, instead of one large enough for a double track. By careful estimates, the first was found to be no more expensive than the other. Besides, for some years, one track would probably be sufficient, perhaps long enough for the saving of interest to build another—at least to go far towards it. This size of tunnel was adopted to give room enough for the largest Pullman car on either of the connecting roads.

The circular form was adopted, because of its greater safety in case a soil should be met with disposed to swell like the London clay. Mr. Storrow, in his Hoosac Tunnel Report, describes a case of this kind, in which, after two attempts to line a tunnel of the usual or horse shoe form, with masonry, the circular form was finally adopted. The lining is to be of brick masonry, laid in cement, generally twenty-four inches thick under the river and eighteen inches on land.

It is proposed to place the two tunnels so far apart under the river, that in case the second one should be delayed till after the completion of the first, an accidental cave would not endanger the stability of the one in use.

Experience in the construction of the river tunnels in Chicago teaches not only the convenience, but the necessity of a drainage tunnel for at least half the distance across the river, in a work like this. It has been thought best to make it entirely across the river for the Detroit tunnel, and for the following reasons:

First.—It would be of great convenience and value, if much water should be encountered in the construction of the main tunnels.

Second.—It would serve a very valuable purpose as a reservoir, should the machinery for pumping out the main tunnels happen to be disabled at the commencement of a great flood of rain falling in the open approaches at the ends of the tunnel, an occurrence that has taken place already more than once in Chicago.

Third.—As the main tunnels must necessarily be curved part of the way under the river on each side, the drainage tunnel, after its completion, would form a most satisfactory base of alignment.

Fourth.—The shafts at each end of it could be made to serve as working shafts for the main tunnels during their construction, and for ventilating purposes afterwards, should that be found necessary.

Fifth.—A reason that did not occur originally, but is now quite apparent; it would afford the means of shortening the time necessary

to complete the main tunnels under the river, and consequently the whole work one-half. This could be done by commencing operations at two points under the river, equidistant from each other and from the shore shafts. In this way, six working faces instead of two could be obtained, so that an average progress from each face of but two feet a day would insure the completion of the work in something less than a year. Of course, on each side of the river as many shafts could be sunk as might be found necessary to complete the land portion of the work as soon as that under the river.

The open approach on the Detroit side fortunately occurs in a wide avenue, and is situated between two streets, so that very little, if any, inconvenience will result to the public travel. It will be twenty-six feet wide, with vertical retaining walls on each side. The Canada approach occurs in an open plain, and will be a simple earth excavation, except at and near the portal, which is to be in fifty-seven feet depth of cutting.

The estimated cost of the whole work is $2,650,000. This estimate was based upon the supposition that no machines would be used in removing the earth except to hoist it up the shafts. It was thought, however, that machinery operated by compressed air could be used, and diminish considerably the cost of the work. If one quarter of the advantages claimed in the *London Times* of January 18th for Brunton's machine, now seriously proposed for boring under the Straits of Dover, can ever be realized, the cost of tunneling through firm ground will be greatly diminished.

It was recommended that the drainage tunnel be constructed before soliciting proposals for the main work, so that contractors might feel more certain of the nature of the ground, and thus be disposed to make lower bids than they otherwise would. This recommendation has been adopted by the companies chartered both in Michigan and in Canada.

Both shafts have been sunk to the bed rock. On the Detroit side this reaches to the bottom of the sump, or about eight and a half feet below the bottom of the drainage tunnel. On the Canada side the bed rock is only two feet below the bottom of the drainage tunnel, and the sump will be sunk a few feet in it. From the Detroit shaft the drainage tunnel has been carried a little more than 600 feet out under the river, and from the Canada shaft a little less than 100 feet, or about

700 feet in all. The Canada shaft was not commenced for several months after the Detroit one was begun.

The rate of progress in this work has thus far averaged not more than one half that which was expected, owing to the very hard ground, composed partly of boulders. On neither side of the river, however, does the layer of boulders extend so high up as the bottom of the main tunnel, and only a few feet of its lowest portion will be in hard-pan. The ground passed through in each shaft shows a great correspondence in position and character, from which it is inferred that the excavation of the main tunnels, under the river especially, will be through a stiff but easily worked blue clay. The fact that a seam separating two slightly different kinds of earth has been in sight most all the way thus far from the Detroit shaft confirms the belief that across the river the different layers of earth, except when cut away by the stream, are uniform in thickness down to the hard-pan, and dip slightly towards the south.

The principal precautions deemed important in the prosecution of the main work are these:

First. To require the contractors to have constantly a force composed, in part at least, of skilled laborers accustomed to mining in difficult ground, should it be met with.

Second. The use of a skeleton shield of iron or steel, which in case of necessity could be fitted out in front and around the sides of the excavation in advance of the masonry.

Third. Frequent probings or borings in front and on the sides, to give warnings of the proximity of bad ground. Even should there be no difficult ground to pass through, these borings would probably prove a safeguard in drawing off large volumes of gas gradually, and thus prevent serious, if not fatal, explosions. This course was found advisable in the construction of the Chicago Lake Tunnel.

Fourth. The use of a large tarpaulin on the bottom of the river immediately over the workmen. This was successfully used, if my memory is correct, (the late fire in Chicago destroyed my books of reference,) in passing a breach that had been filled with bags of sand in the Thames Tunnel, and it is believed could be equally useful in preventing a breach. It would require the employment of divers, who could also be very usefully occupied a great part of the time in probing the bottom of the river just in advance of the work to discover any deep pits or pockets of sand or silt that might exist.

The question has often been asked if it would not be cheaper and better to construct a bridge across the river at Detroit than to make a tunnel under it? Some engineers have looked into the probable cost of a bridge, and believe it would be preferable both on account of the greater economy in cost of construction and in maintenance afterwards; but this question has been set at rest for the present by the absolute refusal of the Canadian Parliament to grant a charter for a bridge so long as a tunnel is considered practicable.

Mr. Charles H. Fisher: I would ask Mr. Chesbrough if he would favor the use of the same tools and methods with which he made the borings?

Mr. Chesbrough: The borings were made with a wood auger,* such as I used in the construction of the lake tunnels here in Chicago, with which specimens from different depths were brought up. Of course, there was doubt whether these always came from the depth to which the auger was sent down, but we found afterwards that it was so in every case. The material which was drawn up with the auger came from the point where the auger ceased to be turned round. In Detroit we have practically relied upon this; but where, however, there is doubt, it would be best to sink a tube, such as is put down for artesian wells; but little of which has been done. The auger we used was one and a quarter or one and a half inches in diameter, and is large enough if the joints are properly made. A very good quality of iron is required to withstand the torsion.

A Member: Will the material always stick to the auger if hauled through water?

Mr. Chesbrough: Not if it is very soft. In such cases we used a tube made of wrought iron; but when the auger stopped in stiff clay we could rely upon it.

A Member: Did you have to remove the auger frequently?

Mr. Chesbrough: We generally drew it up once in about 10 feet, and it usually brought up the whole strata. In some instances we could not remove more than 6 inches of the strata, but it was very seldom.

A Member: I understand that you bored that way to a depth of 90 feet?

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* Like that commonly employed for boring wood.

Mr. Chesbrough: Yes; sometimes 80 or 90 feet were bored in a little over a day; again, to get a satisfactory boring it would take a week. Occasionally a boulder would be hit and the rods broken. Of course the rod was very long, so that one would think it would break in turning, but it did not.

Question: If you struck a boulder rock you would not succeed?

Mr. Chesbrough: Of course not; boulders would stop progress in that place, but we would try in another.

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AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

LOADS AND STRAINS OF BRIDGES.

A paper presented by JOHN GRIFFEN and THOMAS C. CLARKE, Civil Engineers, Members of the Society.

How to obtain uniformity of strength is the problem to be solved by the design of iron railway bridges. The strength of the weakest bridge, and of the weakest part of that bridge, measures the strength of all the bridges on a line of railway. The breaking of a single floor beam may wreck a train and kill and wound many persons ; and it is no consolation to know that all the other floor beams, tie rods, etc., of other bridges of the same line have a superabundance of strength.

The strength of a bridge results from the following conditions :—

The heaviest loads to which it can be subjected :—

The maximum strains resulting from those loads :—

The sizes of the tensile and compressive members, and hence their strains per square inch of area:

The available strength of those members depending upon—First. The quality of the iron of which they are made. Second. The cross-section of the struts. Third. The mode of forming the connections.

Errors of design have been made in respect to all these points.

First. A uniform load per lineal foot has been assumed for all

spans, short and long alike, while the actual load is greater for short and less for long spans, and is always in excess of the general load upon certain parts, such as floor beams.

Second. No distinction has been made between the effects of the dead load of the structure and the moving or live load of trains, suddenly applied and accompanied by shocks and vibrations.

Third. The margin of safety between the allowed strain and the disabling limit of the iron has been overestimated, as the margin of safety of the weakest part measures that of the whole.

Fourth. Sufficient distinction has not been made in specifications between a tough and elastic iron, and a hard and brittle quality, if the ultimate breaking strength of both were alike.

Fifth. The strains allowed upon compressive members are not based upon any definite knowledge of their ultimate powers of resistance.

These points will be considered in turn, and suggestions will be made toward a practice which shall result in uniformity of strength in all lengths of span, in all parts of every span, so that one part shall not give way before another.

The standard of strength must finally be determined by the engineer for each particular case. It would be useless to lay down any rules upon this point. Each man must be free to settle it for himself. But when he has decided it, and says, "I will adopt a margin of safety of three, four, five or six," as the case may be, he wishes to feel certain that all his spans, and all their parts, form no exception to this rule. Uniformity of strength will then be attained; how much strength to give will be always an open question.

I. What are the actual loads to which railway bridges are subjected?

In table No. I., accompanying this paper, will be found a list of the weights and dimensions of the principal types of locomotives now used upon American railways, divided into three classes.

The first includes those engines of exceptional dimensions and weights which are used for pushing trains up heavy grades. Fortunately, their speed is slow.

The second class includes heavy freight and coal engines, whose average speed is ten to twelve miles an hour.

The third class the common form of four driver passenger engines, which cross bridges at from twenty to fifty miles an hour.

Class four contains the various kinds of cars—passenger, freight, and coal.

The following points may be discovered from inspection of this table :—

That the weights of engines and loaded tenders average from 2300 to 2700 pounds per foot of track occupied, and that the weights of tenders separately are but little less.

That owing to the concentrated weight of engine over drivers, the loads carried by spans of less than 100 feet will exceed these weights. As there are so many different types of engines we must select one of average dimensions and weight, leaving provision to be made for the passage of exceptionally heavy engines in the margin of safety which is to be fixed by the engineer of the bridge.

Take, therefore, an engine whose total weight with loaded tender is 125,000 pounds, occupying with pilot fifty feet of track, $\frac{125,000}{50} = 2500$ pounds per foot: distance occupied by wheel base of engine and tender alone is 41½ feet, $\frac{125,000}{41.5} = 3000$ pounds per foot; distance occupied on track by the concentrated weight over drivers, say 17 feet and weight 60,000 pounds, $\frac{60,000}{17} = 3530$ pounds per foot; if the driving wheel base is 15, $\frac{60,000}{15} = 4000$ pounds per foot; if the driving wheel base is 12 feet, $\frac{60,000}{12} = 5000$ pounds per foot. This will give us the following loads :

Spans 12 feet and under	5000 pounds per foot.
" 12 to 17 feet	4000 " " "
" 17 " 25 "	3500 " " "
" 25 " 83 "	3000 " " "
" 83 " 110 "	2500 " " "

Floor beams under 12 feet apart, and track stringers less than 12 feet long, will carry 5000 pounds per foot.

Floor beams 12 to 15 feet apart, and track stringers 12 to 15 feet long, will carry 4000 pounds per foot.

Inasmuch as the weight per foot of cars is considerably less than that of engines, in spans of over 100 feet the actual load per foot will diminish with the length of span.

These results have been arranged in Table No. II., showing for different spans the weights caused by—

1. All locomotives.

2. Reading coal cars drawn by two Reading standard coal engines.

3. Same cars by one similar engine.

4. Pennsylvania box freight cars drawn by two standard freight engines.

5. Same cars by one similar engine.

6. Pullman palace cars drawn by one New York Central passenger engine.

These are the maximum loads which can come upon the chord systems of any of the forms of girder truss, upon the primary system of a Fink truss, or upon the arch and chord of a bowstring. Owing to the excess of weight of the locomotive above that of cars, the loads upon the panel systems of girder, trusses and bowstrings, and the subsidiary systems of the Fink truss will be in excess, and should be taken for all spans at not less than 3500 pounds per foot.

II. It has been stated that it is not customary to make any distinction between the effects of the dead load of the bridge and the live load of trains. This varies very much in ratio according to the length of span. Table No. III. shows what the ratio of dead to live loads is for different spans.

There can be no doubt but that the short spans, where nine-tenths is live load, accompanied by vibration, are more severely strained than the long bridges where half the load is quiescent. It would appear that the margin of safety ought to be greater upon short than upon long spans, in order to give uniform strength.

It is difficult to say what the exact difference is between the effects of dead and live loads. Professor Macquoin Rankine, a very high authority, states in his "Applied Mechanics," "a suddenly applied force is equivalent in strain to twice the same force gradually applied."

This conclusion is confirmed both by the experiments made by order of the English Commissioners upon the application of iron to railway structures so far back as 1849, and by the later experiments of Fairbairn, which will not be quoted here in detail, as they are to be found in all the books.

From them it appeared that a tensile strain of six tons per square inch applied to the bottom flange of a riveted plate girder, and accompanied by vibrations made to resemble as much as possible those caused by a passing train, did not break the girder, although repeated over three millions of times. But when the strain was increased to eight tons per square inch, it broke after 300,000 further applications.

As the breaking strength of average English plate ranges from twenty to twenty-two tons, it would appear that the effect of live load was more than twice as severe as dead load. It is to be regretted that Dr. Fairbairn did not have the girders made of exactly the same dimensions and of the same iron, ascertain the breaking static weight of one, and then apply one-half of this as live weight, and see how many applications it would bear before breaking.

If we agree with Rankine and Fairbairn that the destructive effect of a live load is double that of a dead load, our course is clear. A suggestion, originally made, it is believed, by Unwin in his treatise upon iron bridges, points the way to a simple solution of the problem. Multiply the live load by two, and add it to the dead load. Their sum will be a load which may safely be treated as an all dead load, and a strain per square inch and margin of safety used such as is proper for dead loads.

Table No. IV. shows the equivalent dead loads applicable to all spans. If these loads, or rather this principle of fixing loads, be adopted by engineers, one uncertain element will be eliminated from the problem, and the only point left open will be what limit of strain to put upon the iron.

III. It has been stated that the value of the factor or margin of safety is commonly overestimated. It is not uncommon to read in specifications that the factor of safety shall be *six*, meaning that the working strain shall be one-sixth of the ultimate breaking strain.

A little consideration will show that the true margin of safety is the difference between the working strain and that strain which would give the iron a permanent set and unfit it for use, either by crippling the compressive members or by stretching the tension members so that the bridge would become distorted and "sag" below a level line. Even before this point was reached, the iron in tension would have become "overstrained," causing its particles to suffer permanent derangement. Although this "set," as it is called, does not diminish the ultimate capacity of the iron to support a dead load, yet, as has been pointed out by Stoney, when the "stretch" is taken out of an originally tough piece of iron, it becomes brittle. It is well known that a chain that has been overstrained in testing is liable to snap off with less than its proof load.

This limit of elasticity of wrought iron under tension is that point at which the elongations cease to be in uniform proportion to equal ad-

ditions of load, and coincides very nearly with the point at which visible set takes place. It does not vary much from one-half of the ultimate strength of the iron. Common English plate, bar and angle iron of an ultimate strength of from twenty to twenty-two tons per square inch, has an elastic limit of not over ten tons per square inch. The highest grades of English and American double refined bar iron of an ultimate strength of 55 to 60,000 pounds per square inch, have an elastic limit of from 25 to 30,000 pounds per square inch ; hence a working strain of 10,000 pounds per square inch, gives an available margin of strength or safety, or whatever term we may prefer to call it, of from two to three, instead of six.

Whatever the engineer selects, it should be enough to allow for—1. Possible inequality of material. 2. Imperfection of workmanship. And 3. The effects of deterioration, arising both from use and from natural causes.

A dread of inequality of material is the reason why engineers prefer wrought iron to cast iron or to steel for the construction of bridges. If the engineer could always depend upon getting such a quality of cast iron as the late General Rodman made for artillery, which was worked up to a tensible strain of 27,000 pounds per square inch, and was really more like cast steel than iron, his objections to the use of cast iron would vanish.

It has been stated that in experiments upon the material for the St. Louis bridge, some steel bolts, $5\frac{3}{4}$ inch diameter, broke with 30,000 pounds per square inch, and no elongation ; while small bolts $\frac{3}{4}$ inch diameter, of the same material, bore 100,000 pounds per square inch, and elongated considerably.

Imperfection of workmanship should not be found in our American bridges which are made by machine tools. In riveted lattice and plate girders it is a serious cause of the actual strength falling below that given by calculations. Giving a margin of strength beyond what seems to be required, is, as we have stated, a recognition of the fact that iron bridges will decay like all human works.

But it is not so generally known that if a bridge has not enough iron in certain parts, although built of good iron and put together strongly, it will *wear out* under a heavy traffic, just as locomotives, cars and rails wear out. One or two instances will illustrate this. Where pin connections are used, owing to the concentration of strains which comes upon a pin, it is necessary to "reinforce," as it is termed, the

plates of iron upon which the pins bear, and thus increase the bearing surface until the pressure is reduced to 7 or 8000 pounds per square inch, or else the pin will cut into the iron, or the iron into the pin.

In the Crumlin viaduct, as originally built with pin connections, this principle was not recognized, enough bearing surface was not given, and the pin holes became enlarged. The pins were removed, and the struts riveted to the chords, and this example is frequently quoted to show the superiority of riveted over pin connections, while in reality it only shows imperfect design.

Another still more striking example can be found nearer home. On the Reading railway, plate girder bridges of 25 feet span and under, were originally proportioned for a rolling load of two tons per foot of track. It was found that under the heavy traffic of that road, the webs of these girders at the delivery end crushed or buckled. They have since been rebuilt, or strengthened and proportioned for a rolling load of four tons per foot of track, and now wear very well.

IV. Whatever be the adopted margin of safety, it would appear that a larger margin should be allowed in the case of hard and brittle iron than in that of a tough and ductile quality. But this is just what most bridge specifications do not do.

The experiments of Kirkaldy have clearly shown that a high ultimate breaking strength may be due to the iron being tough, or merely to its being hard and unyielding. In the former case, it will "draw down" and stretch considerably before breaking; in the latter it will snap short off with but little elongation and contraction of area at the point of fracture. One is tough, the other is brittle, and yet both may have an equally great ultimate strength. How shall we know them apart?

The required iron should not be too soft, the limit of elasticity should not fall below 25,000 pounds per square inch before showing visible set. The breaking strength should run from 55 to 60,000 pounds per square inch. A bar a foot long, and of one square inch area, should elongate at least fifteen per cent. before breaking.

As it is not always easy to measure accurately the contracted area at the point of rupture, there is no simpler nor better mode of testing ductility than by bending the bar cold, and such a bar should bend double cold without any signs of fracture.

Mr. G. Berkeley, in his valuable paper read before the London Institution of Civil Engineers at the session of 1870, states his expe-

rience with English irons as follows :—" Experience extending over twenty years, and comprising many thousands of experiments, has proved that a quality of iron can be obtained at the current prices of the day, which will bear the following tests :

"For plates, an average breaking strength of 20 tons per square inch, and a minimum of 19 tons per square inch, and an average stretch of 1 inch in twelve lineal = 8.33 per cent.

" For angle and T irons, an average breaking strength of 22 tons per square inch, and an average stretch of $1\frac{1}{4}$ inches in 12 lineal = 10.5 per cent.

"For rivet iron, an average breaking strength of 18 tons per circular inch."

Common American bar iron will not ordinarily bear over 50,000 pounds ultimate strength, will not elongate over $8\frac{1}{2}$ per cent, and will show signs of fracture when bent cold over 45 degrees.

The undersigned have tested iron as brittle as this, and quite unfit to go into a bridge, the breaking strength of which was over 60,000 pounds per square inch.

Engineers should provide such tests in their specifications as will distinguish the two sorts apart, and if they admit the use of the lower grade iron, should discriminate by fixing a larger margin of safety than for the tougher and better iron. If they do not, they will be pretty sure to get the poorer quality, as it costs less money, and the reason why will be shown.

The mode of making refined iron at Phœnixville is to take a high quality of grey forge pig iron, and work it in a furnace by the process technically known as "boiling," the boiling furnace being "fettled" with ore. This pig iron when "brought to nature," is balled up in the furnace in the usual way, squeezed in a Burden squeezer, and then rolled into a flat bar, technically known as a "muck bar," or No. 1 bar.

From each heat so made, one bar is taken and bent to an angle of 45 degrees cold ; if it stands without any signs of fracture, the heat is passed as good, if not, it is rejected.

The iron that has passed this test is piled, charged in a heating furnace, heated and rolled into flat bars. This is called No. 2 Bar, and is sold as "Phœnix Best." The iron so rolled is again cut, piled, and rolled into the finished bar, and is called No. 3 Bar, and is the iron sold by the Phœnix Iron Co. as "Phœnix Best Best. A bar of this

iron, $2\frac{3}{8}$ inches diameter, has been bent cold so that the sides came in close contact without showing the least signs of fracture.

It should be borne in mind that the object of re-working iron is to refine it by getting rid of the surplus cinder and scoria, making the iron firm in texture and of a more uniform quality. This uniformity of quality results from the fact that the pile from which a bar of No. 2 is made, consists of fourteen No. 1 bars, and the pile of No. 3 of eight No. 2, so that if by chance an inferior muck bar had been used, it would form but $\frac{1}{112}$ part of the No. 3, or "Best Best" bar.

All iron improves up to the third working, but if the quality of the pig is not suitable, no amount of working will make the product good iron, hence the necessity for tests as to toughness and stretching.

The ordinary iron of commerce i made, as a rule, from an inferior quality of pig, is frequently worked in its conversion from carbonate to metallic iron, by the process practically known as puddling, instead of boiling, and is only once worked from the puddle or muck bar, corresponding to No. 2 iron.

It is also made sometimes from scrap iron and often from old rails. Neither of these modes gives reliable iron, as there is no certainty of the quality of the scrap used, though bar iron made from scrap is ordinarily reckoned as good quality. Iron from old rails is always inferior and not to be trusted for the uses of a high grade iron, as rails are generally made in the first place of inferior iron.

Hence it follows that a reliable iron for bridge purposes should be made of a known quality of pig, worked in the best way in the boiling furnace, tested in the muck bar, and cut, piled, heated and rolled once or twice thereafter, according as single or double refined iron is needed.

It is not to be expected, nor is it desirable, that the engineer should dictate the process of manufacture, but he should establish such tests in his specification as will distinguish an inferior from a high quality of iron, and what these tests should be, has been previously stated.

V. But little time remains to speak of the last point. It has been stated that the strains allowed in practice upon the compressive members of iron bridges are not based upon any definite knowledge of their powers of ultimate resistance. The fact, unhappily, is too well known and admitted to allow of argument.

It was the intention of Mr. Samuel J. Reeves, President of the Phœnix Iron Co., and a member of this Society, in conjunction with

the undersigned, to undertake a series of experiments which would have thrown some light upon this important point, and to submit their results at this meeting. The time, however, was found too short to make these experiments as they should be made. They will, however, be made during the coming summer and reported to this Society at or before its next annual meeting.

It is proposed to test to breaking, under a compressive strain, full sized specimens of the shapes of struts commonly used in upper chords and posts of iron bridges. They will be arranged, so far as possible, to have the same connections at top and bottom as in actual use.

The comparative value of the different forms will then be known, and it is hoped that constants can be established for Gordon's formula which will make it possible to ascertain the strength of any length and diameter of wrought iron strut.

TABLE No. I.

ACTUAL WEIGHTS OF ENGINES, TENDERS, CARS, &c.

No.	DESCRIPTION.	Number of Driving Wheels.	Number of Truck Wheels.	Concentrated weight on Drivers divided by length of driving-wheel base.	Resulting Weight per foot.	Total weight of Engine and Loaded Tender divided by distance covered on track, including pilot.	Resulting Weight per foot.
	CLASS No. 1.—"PUSHERS."						
1	Reading Railway Tank, all............	12	None.	102,000 / 19 ft. 7 in.	5204	102,000 / 36 ft.	2833
2	do. do. wit : tender,	10	"	82,200 / 15 ft. 8 in.	5268	132,200 / 54 ft. 1 in.	2448
3	Pennsylvania Railway, " "	8	"	80,000 / 22 ft.	3636	140,000 / 54 ft.	2595
4	Baltimore & Ohio Railway, " "	8	2	84,000 / 12 ft. 6 in.	6720	128,000 / 53 ft.	2415
5	Fairlie double-ender..	12	None.	60,480 / 8 ft.	7560	120,900 / 52 ft.	2326
	CLASS No. 2—HEAVY COAL AND FREIGHT.						
6	Chicago, Burlington & Quincy, freight,	6	4	72,000 / 12 ft	6000	128,000 / 53 ft. 6 in.	2392
7	Reading, standard coal........	6	4	53,000 / 9 ft. 6 in.	5578	122,128 / 50 ft. 3 in.	2430
8	Pennsylvania, standard freight........	6	4	54,500 / 12 ft. 5 in	4360	129,900 / 54 ft.	2405
9	Del., Lack. & W., " "	6	4	71,500 / 12 ft.	5948	138,900 / 54 ft.	2572
10	New York Central, special freight.....	6	2	65,000 / 15 ft. 6 in.	4193	120,000 / 45 ft.	2666
11	Erie broad gauge, " "	6	4	72,156 / 14 ft. 6 in.	4976	137,444 / 54 ft.	2545
	CLASS No 3.—MIXED PASS. AND FREIGHT AND PASS.						
12	Reading, mixed pass. and freight......	4	4	41,440 / 6 ft. 6 in.	6376	115,184 / 45 ft. 7 in.	2526
13	do. standard pass...............	4	4	25,264 / 6 ft. 6 in.	3887	103,260 / 43 ft. 10 in.	2325
14	Pennsylvania " "	4	4	45,400 / 8 ft.	5675	125,300 / 53 ft. 6 in.	2342
15	G. Trunk of Canada, stand. pass. & ft.	4	4	40,320 / 7 ft. 6 in.	5376	112,000 / 49 ft.	2275
16	New York Central " " "	4	4	40,000 / 7 ft. 6 in.	5460	100,000 / 44 ft.	2272
17	Average of loaded tenders............		8	16,500 to 25,000 / 4 ft. 6 in.	3666 to 5550	33,000 to 50,000 / 20 ft.	1650 to 2500
	CLASS No. 4.—LOADED CARS.						
18	Penn. Railway, sleeping and pass. cars,					57,000 / 64 ft. 2 in.	890
19	do. do. box freight "					42,000 / 31 ft.	1355
20	Reading, long coal "					40,000 / 22 ft.	1818
21	Lehigh Valley, short coal "					19,000 / 13 ft.	1461
22	Pullman palace and sleeping "					71,600 / 75 ft.	954

TABLE No. II.

WEIGHT IN POUNDS PER FOOT RUN OF TRACK, FOR DIFFERENT SPANS AND KINDS OF TRAINS.

Length of Spans in Feet.	1. All Locomotive Engines.	2. COAL TRAIN. Cars (No. 20) drawn by 2 Engines, (No. 7.)	3. COAL TRAIN. Cars (No. 20) drawn by 1 Engine, (No. 7.)	4. FREIGHT TR IN. Cars (No. 19) drawn by 2 Engines, (No. 8.)	5. FREIGHT TRAIN. Cars (No. 19) drawn by 1 Engine, (No. 8.)	6. PASSENGER TRAIN. Cars (No. 22) drawn by 1 Engine, (No. 16.)
Under 12	5000					
12 to 17	4000					
17 to 25	3500					
25 to 83	3000					
83 to 110	2500					
110		2430	2094	2405	1870	1481
125		2365	2067	2262	1809	1418
150		2275	2026	2111	1740	1340
175		2200	2000	2065	1710	1285
200		2130	1974	1922	1665	1244
225		2100	1950	1864	1631	1211
250		2068	1943	1809	1603	1186
300		2026	1922	1733	1562	1147
350		2000	1907	1679	1532	1120
400		2000	1895	1638	1510	1100

TABLE No. III.

Ratios of Dead to Live Load, for Different Spans.

Length of Spans in Feet.	Dead LOAD of Bridge Track, Rails, &c., per foot.	Live LOAD of Coal Train with two Engines, per foot.	TOTAL LOAD, lbs., per foot.		RATIO OF DEAD TO LIVE LOADS.	
					DEAD.	LIVE.
Under 12	500	5000	5500		.09	.91
12 to 17	550	4000	4550		.12	.88
17 to 25	625	3500	4125		.15	.85
25 to 50	700	3000	3700		.19	.81
50 to 83	800	3000	3800		.21	.79
100	900	2500	3400		.26	.74
110	1000	2430	3430		.30	.70
125	1135	2365	3500		.32	.68
150	1225	2275	3500		.35	.65
175	1300	2200	3500		.37	.63
200	1500	2130	3630		.41	.59
225	1700	2100	3800		.45	.55
250	2000	2068	4068		.49	.51
300	2400	2026	4426		.54	.46
350	3000	2000	5000		.60	.40
400	4000	2000	6000		.66	.34

TABLE No. IV.

DEAD AND LIVE LOAD PER FOOT REDUCED TO EQUIVALENT DEAD LOAD.

1. Length of Spans in Feet.	2. Dead LOAD of Bridge, &c., per foot.	3. Twice Live LOAD of Coal Train per foot.	4. Sum of Columns 2 and 3, being equivalent Dead Load per foot.
Under 12	500	10,000	10,500
12 to 17	550	8000	8550
17 to 25	625	7000	7625
25 to 50	700	6000	6700
50 to 83	800	6000	6600
100	900	5000	5900
110	1000	4860	5860
125	1135	4730	5865
150	1225	4550	5775
175	1300	4400	5700
200	1500	4260	5760
225	1700	4200	5900
250	2000	4136	6136
300	2400	4052	6452
350	3000	4000	7000
400	4000	4000	8000

XLIV.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.*

EXPERIMENTAL STRAINS UPON A BOWSTRING TRUSSED GIRDER.

A paper presented by GEN. THEO. G. ELLIS, Civil Engineer, Member of the Society.

While recently engaged in investigating some graphic methods for obtaining the true strains in a certain form of bowstring trusses, a series of experiments upon the diagonal bracing was made, which, it is thought, may prove interesting to the profession as a test of the mathematical methods of computing the same strains.

The experiments were made upon a carefully constructed model, free to turn at all points of attachment, so that it possessed no stiffness except that due to the bracing and to the very small amount of friction at the joints. This construction was adopted so as to get results which should agree with correct theoretical computed strains, rather than those occurring in actual practice; where an indeterminate amount of strength and stiffness is given to the truss by the rigidity of the points of connection and the continuity of the arch and chord.

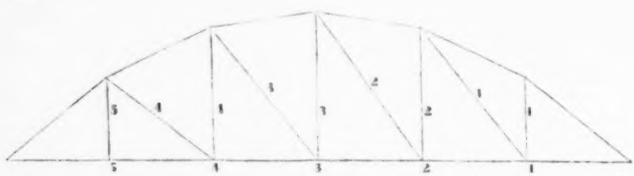
The only experiments known to the author, that have been tried upon the bracing of bowstring trusses, are those of Mr. W. Airey,

* Announcement of this paper was accidentally omitted at the Convention.—[G. L.]

described in a paper read before the British Institute of Civil Engineers, in April, 1868, and some experiments on bowstrings and braced girders by Mr. Charles F. Gower, described in Engineering of May 20th, 1870.

The former were made upon a rigid truss, with a continuous steel bow and a wooden string, with all the posts and braces in place at the same time, inducing complex strains in more than one system for the same load. They are useful as showing what the strains would be in a truss of exactly that construction, but do not appear to be capable of generalization for other trusses.

The strains were determined by the musical tone of the wires forming the ties, when compared with that of a separate wire of the same vibrating length, loaded so as to give the same pitch. The compressions were measured by the differences of tension between two different loads.



The latter experiments above mentioned are more useful. They were, however, made upon a circular arc, with fixed abutments, and, as published in Engineering, are not sufficiently explained. The diagrams giving the strains are also somewhat unintelligible.

The model used in the present experiments was 24 inches in length, divided into six bays of 4 inches each, between working points. Between the bays were five vertical posts, of the heights of 2, 3.2, 3.6, 3.2, and 2 inches, respectively, making the top points and the ends of the truss in the line of a parabola. From the bottom of each post to the top of the next was a diagonal brace as shown in the diagram. The parabolic form was taken for the top, or compression member, to eliminate all strains upon the diagonals in the unloaded or uniformly loaded truss.

The weight of material in all the bays being made equal, hinging equal weights upon the points of support, the whole system was in equilibrium without any strain upon the diagonals when unloaded, or loaded equally at each point of support.

Although using a circular arc would not probably have materially

affected the results, yet using the parabola, equally loaded by the weight of the truss, removes all uncertainty relative to the amounts of the strains transmitted by variable loads.

The compression member was made of pine wood, about $\frac{5}{8}$ by $\frac{1}{4}$ inch, carefully jointed so as to turn freely at the top of each post upon an ordinary brass pin, with as little friction as possible. The rest of the frame was made of a stiff card-board, and secured at the joints by a single pin passing through them. The sections of the chord had a small block of wood at the foot of each post, through which the pin passed to keep it in position. Thus every part was in equilibrium, but was free to turn if not held in place by the tension or compression of some part of the truss.

The card-board parts of the framing were attached to one side of the upper or compression member, which was the only part having a sensible thickness, the better to measure the strains, as will be described.

The weight of the arch and pins was	.91 oz.
" " " chord " "	.17 "
" " " posts " "	.20 "
" " " braces " "	.12 "
Total weight of truss........................	1.40 "

This truss was suspended by the ends by vertical threads, so as to hang freely, and was kept from overturning by horizontal guys from the top of the arch. The whole was thus free from any friction on abutments, or anything that could affect the strains in the truss itself.

While in the position above described weights were suspended from the lower ends of the posts, which were provided with hooks for the purpose, and the strains upon the posts and diagonals carefully measured.

For measuring the tensions, a bent spring made of steel wire was used. A piece of wire about six inches long was bent in the middle so that the ends would nearly meet. The extreme ends, for about .06 of an inch, were bent out at right angles to the plane of the spring and sharpened. A strain was then applied to separate the points, and a scale prepared showing the distance between the points for each tenth of an ounce from 0 to 2.5 ounces. This was carefully measured, so as to make the bent wire a sensitive spring balance, showing by inspection,

on applying the scale to the points, exactly the strain upon the spring. After the balance was thus prepared, and one or more weights hung upon the truss, one of the braces was cut across and the spring applied by inserting the sharp ends into small holes in the card-board, and altering its position gradually until the severed ends of the member cut would just meet. The scale was then applied and the strain on the spring measured. In this way the tensions on the several parts as they really existed, were estimated with great exactness; the theoretical strains due to the weights being affected only by the slight friction of the joints of the truss.

In measuring compressions on the posts and diagonals, the bars were cut in the same manner. It was, however, found impracticable to measure them directly with a compressive spring balance, as the means taken to keep the parts from bending out of place would create too much friction for correct results. The compressive forces were therefore converted into tensions in the following manner; a slender and light standard of wood was placed so that its foot rested upon the pin through the foot of the post or tie to be measured, and laid in the exact direction of the bar, past the connection with the arch on top. One arm of the tension spring, before described, was then attached to the pin at the upper end of the bar, and a thread attached to the other arm was passed over the top of the standard, through a notch, and could be lengthened or shortened easily, to bring the severed parts into contact by drawing it more or less through the notch.

This apparatus transferred the compressive force on the bar into a tensile force pressing the top of the standard upward from the arch. This latter was measured as tension in the manner before described, with the same spring and scale.

The weight of the spring was .05 of an ounce, and that of the standard .01 of an ounce.

The weights used were all of 2 ounces each, and were hung to the foot of each post in such a manner as to give the strains caused by loading each post separately, and also those caused by loads covering more than one bay, or point of support.

In determining the strains upon the diagonals, the exact strain caused by the load was directly measured, as there was no initial strain on those bars. In measuring the tension and compression upon the posts, however, it was necessary to take into account the tension upon them already existing from the weight of one bay of the chord and half

the adjacent diagonal. When the post was cut, the weight of the lower part of the post was also added.

From the weights of the parts we should have this tensile strain, about .06 of an ounce. In measuring the compression on the posts, a slight additional tension was caused by the weight of the measuring apparatus. The whole initial tension in this case was found by direct trial on the unloaded truss to be about .08 of an ounce. In the experiments this has been assumed to be .1 of an ounce for the correction of the measured strains from the loads.

Friction and other causes rendered the measurement of the strains to within less than .1 of an ounce impracticable, when the truss was loaded.

In some cases, where the strain upon a bar was small, double weights, or 4 ounces, were placed upon each loaded point. In the following tables these strains will be found under the head of "double load."

TENSION STRAINS ON DIAGONALS.

STRAINS ON TIE No. 1.

Points loaded.	Single load, 2 ozs.	Double load, 4 ozs.
1.	2.1 ozs.	..
1, 2.	1.2 "	..
1, 2, 3.	0.6 "	1.2 ozs.
1, 2, 3, 4.	Friction holds.	Friction holds.

STRAINS ON TIE No. 2.

Points loaded.	Single load, 2 ozs.	Double load, 4 ozs.
1.	0.75 ozs.	1.5 ozs.
1, 2.	2.2 "	..
1, 2, 3.	1.05 "	2.1 "
1, 2, 3, 4.	.4 "	1.0 "
2.	1.6 "	..
2, 3.	.35 "	.7 "
2, 3, 4.	Compression.	..

STRAINS ON TIE No. 3.

Points loaded.	Single load, 2 ozs.	Double load, 4 ozs.
1.	.4 ozs.	.75 ozs.
1, 2.	1.2 "	..
1, 2, 3.	2.2 "	..
1, 2, 3, 4.	.75 "	1.5 "
2.	.8 "	1.6 "
2, 3.	1.9 "	..
2, 3, 4.	.5 "	1.0
2, 3, 4, 5.	Compression.	..
3.	1.2 ozs.	..
3, 4.	Compression.	..

STRAINS ON TIE No. 4.

Points loaded.	Single load, 2 ozs.	Double load, 4 ozs.
1.	.25 ozs.	.5 ozs.
1, 2.	.6 "	1.2 "
1, 2, 3.	1.25 "	..
1, 2, 3, 4.	2.0 "	..
2.	.4 "	.8 "
2, 3.	1.0 "	2.0 "
2, 3, 4.	1.9 "	..
3.	.6 "	1.2 "
3, 4.	1.5 "	..
4.	.8 "	1.6 "
4, 5.	Compression.	..

COMPRESSION STRAINS ON DIAGONALS.

STRAINS ON STRUT No. 1.

Points loaded.	Single load, 2 ounces.	Double load, 4 ounces.
5.	.2 ozs.	.4 ozs.
5, 4.	.6 "	1.2 "
5, 4, 3.	1.3 "	..
5, 4, 3, 2.	2.2 "	..
4.	.4 "	.9 "
4, 3.	1.2 "	..
4, 3, 2.	2.0 "	..
3.	.7 "	1.3 "
3, 2.	1.6 "	..
2.	.9 "	1.8 "

STRAINS ON STRUT No 2.

Points loaded.	Single load, 2 ounces.	Double load, 4 ounces.
5.	.3 ozs.	.7 ozs.
5, 4.	1.2 "	..
5, 4, 3.	2.3 "	..
4.	.75 "	1.5 "
4, 3.	1.9 "	..
3.	1.2 "	..

STRAINS ON STRUT No. 3.

Points loaded.	Single load, 2 ounces.	Double load, 4 ounces.
5.	.7 ozs.	1.4 ozs.
5, 4.	2.1 "	..
4.	1.4 "	..

STRAINS ON STRUT No. 4.

Points loaded.	Single load, 2 ounces.	Double load, 4 ounces.
5.	1.8 ozs.	..

TENSION STRAINS ON POSTS.

STRAINS ON POST No. 1.

Points loaded.	Single load, 2 ozs.		Double load, 4 ozs.	
	Observed strain.	Corrected strain.	Observed strain.	Corrected strain.
1.	.8 ozs.	.7 ozs.	1.5 ozs.	1.4 ozs.
2.	.7 "	.6 "	1.3 "	1.2 "
3.	.5 "	.4 "	1.0 "	.9 "
4.	.3 "	.2 "	.6 "	.5 "
5.	.2 "	.1 "	.4 "	.3 "

STRAINS ON POST No. 2.

Points loaded.	Single load, 2 ozs.		Double load, 4 ozs.	
	Observed strain.	Corrected strain,	Observed strain.	Corrected strain.
1.	Compression.	Compression.	..	..
2.	1.2 ozs.	1.1 ozs.	..	..
3.	.8 "	.7 "	1.6 ozs.	1.5 ozs.
4.	.6 "	.5 "	1.2 "	1.1 "
5.	.3 "	.2 "	.6 "	.5 "
3, 4, 5.	1.7 "	1.6 "	..	..
4, 5.	.9 "	.8 "	..	..

STRAINS ON POST No. 3.

Points loaded.	Single load, 2 ozs.		Double load, 4 ozs.	
	Observed strain.	Corrected strain.	Observed strain.	Corrected strain.
1.	Compression.	Compression	..	..
2.	"	"	..	..
3.	1.5 ozs.	1.4 ozs.	..	..
4.	.9 "	.8 "	1.8 ozs.	1.7 ozs.
5.	.5 "	.4 "	1.0 "	.9 "
4, 5.	1.4 "	1.3 "	..	..

STRAINS ON POST No. 4.

Points loaded.	Single load, 2 ozs.		Double load, 4 ozs.	
	Observed strain.	Corrected strain.	Observed strain.	Corrected strain.
1.	Compression.	Compression.	..	..
2.	"	"	..	..
3.	"	"	..	..
4.	1.8 ozs.	1.7 ozs.	..	..
5.	.9 "	.8 "	1.8 ozs.	1.7 ozs.

Post No. 5 sustains only the direct load attached to it; otherwise it has no strain.

COMPRESSION STRAINS ON POSTS.

There is no compression on Post No. 1.

STRAINS ON POST No. 2.

Points loaded.	Single load, 2 ozs.		Double load, 4 ozs.	
	Observed strain.	Corrected strain.	Observed strain.	Corrected strain.
1.	.4 ozs.	.5 ozs.	1.0 ozs.	1.1 ozs.

STRAINS ON POST No. 3.

Points loaded.	Single load, 2 ozs.		Double load, 4 ozs.	
	Observed strain.	Corrected strain.	Observed strain.	Corrected strain.
1.	.1 ozs.	.2 ozs.	.3 ozs.	.4 ozs.
1, 2.	.6 "	.7 "	1.25 "	1.35 "
1.	.3 "	.4 "	.75 "	.85 "

STRAINS ON POST No. 4.

Points loaded.	Single load, 2 ozs.		Double load, 4 ozs.	
	Observed strain.	Corrected strain.	Observed strain.	Corrected strain.
1.	.0 ozs.	.1 ozs.	.1 ozs.	.2 ozs.
1, 2.	.2 "	.3 "	.4 "	.5 "
1, 2, 3.	.4 "	.5 "	1.0 "	1.1 "
2.	.1 "	.2 "	.25 "	.35 "
2, 3.	.3 "	.4 "	.75 "	.85 "
3.	·2 "	.3 "	.4 "	.5 "

There is no compression on Post No. 5.

XLV.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

EXPERIMENTS ON THE DEFLECTION OF CONTINUOUS BEAMS, SUPPORTED AT EQUIDISTANT POINTS.

A paper read by JAMES B. FRANCIS, Civil Engineer, Member of the Society.

In the case in the deflection of beams, "fixed at each end and loaded in the middle," there is a discrepancy in the leading authorities.

According to Barlow, (Report on the Present State of our Knowledge respecting the Strength of Materials, made at the third meeting of the British Association for the Advancement of Science, held at Cambridge in 1833,) the deflection is two-thirds of that in the case, "supported at each end and loaded in the middle." According to Navier, (Resume des Leçons donnees a l'ecole des Ponts et Chaussees, sur l'application de la Mecanique, &c. 2d edition, Paris, 1833), it is one-quarter or three-eighths only, of the deflection according to Barlow.

Not feeling competent to decide which, if either, of these eminent authorities was correct, and having occasion to apply it in practice, I made the following experiments:

A frame was erected, giving 4 bearings in the same horizontal plane, 4 feet apart, making 3 equal spans, each bearing being furnished with a knife edge on which the beam was supported. Immediately over the bearings, and secured to the same frame, was fixed a straight edge, from which the deflections were measured.

EXPERIMENT 1. A bar of "common English refined" iron, marked "J crown K, best," 12 feet $2\frac{3}{4}$ inches long, mean width 1.535 inches, mean depth 0.367 inches, was laid on the 4 bearings, and loaded at the centre of each span, so as to make the deflections the same, the weight at the middle span being 82.84 pounds, and at each of the end spans 52.00 pounds. The deflections with these weights were as follows:

At the centre of the middle span ....0.281 inches.
At the centre of the end spans ....0.275 and 0.284 inches, mean, 0.280 "

The deflections of the 3 spans being, as nearly as practicable, the same, the middle span is in the condition of a beam "fixed at both ends and loaded in the middle," each of the end spans "being fixed at one end and supported at the other." A piece 3 feet $11\frac{1}{2}$ inches long was then cut off from each end of the bar, leaving a bar 4 feet $4\frac{3}{4}$ inches long, which was replaced in its former position and loaded with the same weight (82.84 pounds) as before, when its deflection was found to be 1.059 inches, or 3.77 times the deflection when "fixed at both ends and loaded in the middle."

EXPERIMENT 2. A bar of iron of the same quality and length as in Experiment 1, nearly square, its mean width being 0.553 inches, and mean depth 0.549 inches, was laid on the same bearings, and loaded with the same weights, the deflections being as follows:

At the centre of the middle span ....0.242 inches.
At the centre of the end spans ....0.238 and 0.244 inches, mean, 0.241 "

The bar was then reduced in length as in Experiment 1, leaving 4 feet $3\frac{3}{4}$ inches, which was replaced in its former position and loaded with the same weight (82.84 pounds) as before, when its deflection was found to be 0.983 inches, or 4.06 times the deflection, "when fixed at both ends and loaded in the middle."

The result of both experiments agreed substantially with Navier, who finds the deflection in the case of a beam "fixed at one end, supported at the other, and loaded in the middle," to be $\sqrt{\frac{1}{5}}=0.447$ of the deflection in the case, "supported at each end and loaded in the middle." In the foregoing experiments, the end spans correspond to this case, and the observed deflection with a weight of 52 pounds, were 0.419 and 0.391 respectively, of the deflections in the case, "supported at the end and loaded in the middle," differing somewhat, but not very widely, from the proportion given by Navier.

XLVI.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

FURTHER NOTES ON THE CAISSONS OF THE EAST RIVER BRIDGE.

A paper read by F. COLLINGWOOD, Civil Engineer, Member of the Society.

A paper read before this Society at the last annual Convention, entitled "A Few Facts about the Caissons of the East River Bridge,"* gives account of the construction and sinking of the Brooklyn caisson; these Notes refer almost entirely to the New York caisson.

CONSTRUCTION.

Preliminary borings on the sites of the piers of this Bridge, disclosed the fact that the river bed and underlying strata on the Brooklyn side were essentially different from those on this, the New York side. There at a depth of about 45 feet, a uniformly hard bottom was reached, which was entirely satisfactory as a foundation. Here on the contrary, the strata were exceedingly irregular, all more or less yielding, and with extensive beds of quicksand reaching nearly to the rock, which was found at a depth from 77 to 92 feet below high tide.

It was at once decided, therefore, that the foundation must go nearly

* No. XXX. Transactions.

if not quite to the rock. To provide for the pressure of a larger mass of masonry, compared with that of the Brooklyn pier, it was deemed advisable to increase the size of the caisson, and it was made 102 by 172 feet; the air-chamber was of the same height as in the other caisson, 9 feet 6 inches. The cross-frames, for the support of the central part of the roof were much heavier, being of solid timber, 4 feet thick ; and additional bearing surface on the quicksand was obtained by the introduction of minor frames both across and lengthwise of the caisson. The total sustaining surface, when all brought to a bearing, was about 2,700 feet, or fully twice that of the Brooklyn caisson. This, it was thought, was required to guard against the dangerous settlement that might occur should there be a sudden escape of air from the air chamber, which previous experience had shown to be quite possible.

The roof was increased in thickness from 15, to 22 feet, and a coffer dam built from the top of the twelfth roof course to high water mark. By this means greater buoyancy was obtained, the pressure on the frames and other parts during sinking, lessened, and the stability materially increased.

LEAKAGE OF AIR.

The interior of the air-chamber was lined throughout with light boiler iron riveted together, and caulked. The surface covered by the lining had an area of about 26,000 square feet and was pierced by 6,000 bolts. Although great pains were taken to make these tight, it was found after launching, that four compressors were required to overcome the leakage of air. Men were sent down, and floating around on rafts, they succeeded in stopping the leaks so that one compressor at half speed and throwing about 90 feet of air per minute, more than overcame the loss.

When the pressure was 34 pounds above the atmosphere, with the shoe sealed by water, the leakage was about three times as great. With the shoe uncovered, the escape of air underneath it and upward through the sand, was frequently 1,200 to 1,500 feet per minute, requiring nearly the whole compressor force to supply it. This could always be partially controlled, by banking up inside the shoe with earth.

THE AIR SUPPLY

was furnished by 13 compressors, throwing 3 cubic feet of air (normal pressure), per revolution. These were seldom all running

at once ; at the greatest pressure, the number varied from 8 to 12, with a speed of 30 to 80 revolutions per minute. The highest limit was of course reached, when the sand syphons were throwing out material. In the compressor room, a mercurial guage showed the actual pressure, and, close by it, an indicator moved up and down with the tide, and traversed a strip of paper, on which were marked pounds. The paper being adjusted each morning, the engineer had always before him the real and the required pressure. At the beginning of the sinking, the pressure was kept fully up to that required to balance the water pressure at the shoe.

After a few feet of excavation, it was found that a half pound less pressure was sufficient to keep the water out. At 20 feet of excavation, or a total depth of 57 feet, we began to run regularly with a pound less. The material, down to this depth, was sand and gravel, but it evidently offered a sensible resistance to the passage of water through it. When the pressure was run down by the excessive use of the sand syphons, the water would run in slowly for an hour or two; the pressure had then to be put up to the highest safe limit, and held there two or three hours, to drive the water out.

THE WATER SHAFTS.

One inconvenience arising from this, was the difficulty in keeping a safe supply of water in the pools around the water shafts. The natural effect of the air pressure would be to force the water deeper down in the centre of the chamber (where the shafts were), than at the shoe ; hence there was—as proved by currents in the trenches,—a constant loss by drainage from the shafts to the chambers in addition to that caused by the dredge buckets. The head of water in the city mains being too small to keep up the supply,—as a precautionary measure, a steam pump was connected with the caisson water pipes, which prevented further trouble from this cause.

The water shafts projected 2 feet below the shoe, and the water was always kept in the pools a foot above the bottom of the shafts, and ordinarily much higher. This was necessary to provide against sudden fluctuations in pressure, and on account of the pulsations caused by the plunging in or withdrawing of the buckets.

The sand in the pools always stood at a very low angle, hence they were kept large and well dug out. Unless this precaution were taken, the shafts were liable to be dangerously sealed, when the work was

stopped, by the slow deposit of earthy matter, held in suspension by the water while dredging was being done.

As to be expected, the shafts were most useful during the early part of the work, in removing mud and other materials which could not be taken out by the sand syphons. They were employed, however, until a depth of 74 feet was reached, where they became practically useless, for the reason that, in lifting through such a depth of water, nearly all the material was washed out of the buckets.

CAISSON LEVEL.

Before the caisson was put in place, the river bottom, at a depth of 37 feet, was dredged to as nearly a uniform level as possible. The scour that took place under the outer edge of the caisson before it was firmly grounded, caused it to take the bottom at low tide, fully 2 feet out of level. To prevent this, earth was thrown in at high tide, and a tolerably uniform bearing obtained.

At one time afterward, before all the dock logs and similar obstacles had been cleared from under the frames, the caisson was 9 inches out of level at the corners; but after getting well into the sand, and while settling from 6 to 11 inches per day, the greatest difference of level was 6 inches; and at times for a whole week it would not be out more than 1 or 2 inches. Towards the end of the excavation, fully one-half of the caisson rested on quicksand, and the other half on cemented gravel; but even under these extreme circumstances, we were able to put the hard side down lower, while the soft side remained stationary.

Mr. COLLINGWOOD said: To show how readily the level was controlled, I will state: we stopped the caisson exactly at 78 feet, and the differences of level at the extreme corners of the coffer dam was only an inch, and of the masonry only three-fourths of an inch. We stopped and held it there without difficulty.

SIDE FRICTION.

The excess in weight of masonry, timber, and other parts over the lifting force of the air, varied from about 500 tons to 20,000 tons, the latter being the extreme amount at low tide, when the pressure had been allowed to run 5 pounds below that due to the head of water, and the caisson was at its greatest depth. It was found im-

practicable to determine the side friction with accuracy, for the reason that at no time were the frames and shoe entirely unsupported. Two different observations gave about 400 pounds per square foot; towards the close, when the material was compacted more firmly against the caisson from the slowness of its settlement, the side friction was evidently much greater—probably between 500 and 600 pounds per square foot. To increase the weight, a considerable portion of the sand blown out was allowed to remain inside the coffer dam.

Notwithstanding the large overweight on the caisson (above its buoyancy when inflated), the roof was scarcely changed in form; the depression, as shown by levels, being about an inch;—an admirable test of the thoroughness of the bolting. No blocking was used under the frames, as on the Brooklyn side; they rested directly on the sand.

FOUNDATION.

When within about 10 feet of the highest portion of the bed-rock, a complete series of soundings was made over the bottom. These revealed a stratum of cemented gravel over the entire area, at a depth from 77 to 80 feet, and above and below this a great number of boulders, many of them of large size.

This was the first stratum reached which was of suitable material to sustain the pier, and bore certain evidences of not having been disturbed. For instance, at 62 feet a bone of the domestic sheep was found in the sand under a large boulder; and still lower down, fragments of brick and pottery.

The settlement of the Brooklyn pier on a similar foundation, with two-thirds of the full load upon it, had been but three-eighths of an inch; therefore it was decided to stop excavation at the depth of 78 feet. For about 60 feet on one side, the bed rock was struck at 77 feet; this was blasted out a foot below the shoe, and the space filled in with earth. At two corners a small depth of quicksand remained above the hard stratum; this was removed, and concrete put in its place.

The filling was done as in the other caisson, except that the greater supporting power of the frames rendered the brick piers superfluous, and they were dispensed with. In filling, the spaces under the frames were first made solid; then, wherever there was quicksand under the shoe, a trench was cut out down to the gravel layer mentioned, and filled with concrete. After this the filling proceeded by sections, and bulkheads were put up for the purpose, as described in the paper before mentioned.

While sinking the last 8 feet, all the stones found, except the small ones, (which were removed in the usual manner,) were stowed in pockets attached to the frames, or piled near the centre of the chambers, in spaces excavated for the purpose.

The last 2 feet of excavation consisted mostly of trenching; and as far as practicable, the earth was heaped up and not removed, by which means a large amount of concreting was saved, and the expense of filling lessened.

SAND SYPHONS

have here been mentioned frequently, but no description given of them. In designing the caisson, pipes had been carefully located over the whole area of the roof, with the idea of moving the material excavated the least possible distance. The first plan was to use water as the vehicle by which to carry out the sand, and for this reason the pipes were grouped so as to give one of 4 inches to every 3 or 4 of 3.5 inches diameter. These pipes extended vertically through the roof, and sections were added above as the caisson descended.

During the early part of the excavations, the material was such that it had all to be removed by the dredges. Before it became practicable to employ the syphons, another consideration, namely, ventilation, caused an entire change of plan, and it was decided to use the air of the caisson as the motor. The caisson leaked so little that otherwise, to maintain a proper degree of purity, it was necessary to waste air ; and there was no good reason why the power thus stored up should not be utilized. This being decided, the question of interior arrangement came up. We first tried rubber hose lined with spiral springs to prevent collapsing, making the pieces about 5 feet longer than the chamber was deep, or 14 feet in length. The hose thus hung from the roof in a curve and it was thought that, by sweeping it around, all the material within the circle described could be taken out with little or no shoveling.

From the constant jamming of small stones and sand in the bend of the hose, even when mixed with an abundance of water, this method very soon proved impracticable. We next tried an injector, allowing air to enter an adjustable annular space surrounding a central passage for the sand and air, to which passage a pipe was connected, leading to the bottom of the chamber ; this worked tolerably well, but wasted too much air for the work done.

The next step was to shorten the hose, so that it reached within a few inches of the bottom of the caisson. The material was thrown by about six laborers to a point where another laborer kept the lower end of the hose in moderate motion from side to side. This worked very well, but the sand soon wore out the springs, and the hose collapsed.

The final arrangement adopted was as follows: A piece of pipe 2 feet long was screwed on, near the roof; to this a full way cock of the same size opening was attached, and next a straight piece of pipe extending to within a foot of the bottom of the caisson. The most effective way of working this was to heap the sand and gravel around the lower end of the pipe for 2 or 3 feet in depth; then to open the cock wide and let the air carry the material out.

When working at the best in clean sand, a pipe 3.5 inches in diameter, under a pressure of 25 pounds per square inch, has thrown out 30 cubic yards in an hour. Owing to the large admixture of gravel with the sand, the stoppages from choking were frequent, and this speed was never reached except experimentally. When the pressure reached 28 pounds, it was found that the 3.5 inch openings allowed too much air to escape, and the pipes were reduced at the lower end to 2.5 inches diameter. This arrangement was all that could be desired, so long as the sand was clean. At the last, however, there was a large admixture of clay with the sand, and we had to remove the reducers, and use fewer syphons.

The sand was very sharp, and our next trouble was, that the pipes wore through near the roof. When this happened, a 3 inch pipe was forced up about 2 feet into the 3.5 inch pipe remaining in the roof, and a 3 inch cock attached. When this wore off, a second reduction to 2.5 inches was made. A mechanic was on hand at all times to make such repairs as were needed, and to clear the pipes.

Occasionally, two stones would jam in a pipe at some distance up; these were easily removed by forcing up jointed rods from below, or dropping a bar attached to a rope from above. When working at a pipe from below, the precaution was always taken to screw on a cast iron cap above.

During the first 25 feet of excavation, it was necessary to throw sand over the sides of the caisson, to keep the piling surrounding it from sinking; the remainder was thrown on the dock and saved for future use. As the pipes leading from the air chamber were vertical, elbows were used to change them to horizontal—an arrangement easily

made, but the furious blast of sharp sand cut through in 30 minutes the cast iron elbows at first provided. Wrought iron an inch thick would not stand over 10 hours' use—end wood was bored through as though by an auger.

The plan finally adopted was to use an iron elbow of a foot radius, in which there was an opening on the convex side, in width, equal to the interior diameter, and extending within 2 inches of either end; into this a back piece of chilled iron, about 2 inches thick, was fitted and secured by a key driven into 2 lugs cast upon the elbow. It was the work of a few minutes to remove the key and change one of these back pieces; a man was kept on the top of the caisson to attend to this. The device was much cheaper than any other tried, and worked satisfactorily.

Great care was required to prevent accident from the syphons, as with a heavy presure, small pebbles were discharged almost with the velocity of musket balls. One of the men, by incautious exposure, had a wound made in his arm more than 5 inches long. At the last, when wasting the material inside the coffer dam, the elbows were all removed, the pipes cut down, and a heavy piece of lime-stone placed above each, on the coffer dam braces.

THE TELEGRAPH

used in sending messages up and down was very simple, and worked well. One of the large tubes was capped below, an inch tube, (riveted at the joints to prevent slipping,) was passed through the cap, and indexes attached. Underneath each index was placed a small horizontal table, on which was distinctly traced a plan of the caisson, together with the position of every pipe and shaft. By the revolution of the small tube, therefore, attention could be called to any one of these in a moment.

A small rod, also, was passed down through the large tube, and its weight balanced by a weight above, attached to a cord passing over a pulley. A small index was fixed both above and below; these traversed vertical boards on which were words and sentences such as, "stop," "start," "bucket is caught," &c., directing what was to be done.

LIGHTS.

In the paper before mentioned, I touched upon this subject and suggested a theory as to the reason why all the lights used, (except oxy-

hydrogen) smoked so insufferably. The explanation then given was, that the increased tension of the air diminished the freedom of motion of its particles, and that as a consequence of its imperfect circulation about the flame, the carbonic acid and nitrogen set free, surrounded the flame as a film, caused imperfect combustion, and the elimination of large quantities of carbon.

The burners first used in the present caisson were 5 feet, fish-tail, and burned with a flickering yellow light, consuming a great deal of gas, and smoking almost as badly as candles. It seemed to me that if the gas could be forced out in a thin sheet, under considerable pressure, the difficulty might be overcome. Acting on this supposition, I procured burners of various sizes, and also several kinds of pressure regulators. Experiment soon showed that the regulators were—at least for our use, worthless. But to our great satisfaction, on trying a 2 foot lava tip, of the pattern known as the "excavated bat wing"—it burned with a clear white light, and gave off scarcely any smoke. As the depth increased, these also began to smoke; and when at 65 feet depth, one foot burners of the same pattern were substituted. No farther trouble was experienced.

The arrangement for gas supply was very simple. An iron tank was fixed in the air-chamber of the caisson, which held about 60 cubic feet. Into this the gas was forced by a force pump, stationed on the dock above; and from it distribution made to the various burners. A water tank above had a pipe leading from it to the bottom of the tank in the caisson. The water column thus formed, gave the real measure of the pressure under which the gas was burned. The difference of head (allowing for the difference between salt and fresh water) varied from 2 feet at extreme high tide, to 9 feet at extreme low tide; instead of about 2 inches as is ordinarily the case.

VENTILATION.

Soon after the excavation had been started, some peculiar sensations experienced by the workmen, led to the suspicion that an amount of carbonic acid too large for health was present in the air of the chambers. A rough test with lime water showed this to be true.

The question then arose, how much air was required—we were already forcing down more than stated by many authors as necessary to supply a reasonable number of lights, and the men employed; the amounts given varied from 3.5 to 23 cubic feet per man per minute. It

appeared at once that these estimates had been made under exceeding diverse conditions, and afforded no accurate basis for us to work upon.

There seemed, however, to be more uniformity in the published experiments on the amount of air inhaled by each person; so that it was possible to arrive with some accuracy at the actual amount of carbonic acid given off; knowing the amount of gas burned per minute, the carbonic acid from this source also was readily determined. With the total amount of carbonic acid known, (having decided on what percentage of vitiation of the air would be allowed,) the amount of air required to insure a given degree of purity was found by a simple proportion.

The exhalations from the body are of two kinds; those from the lungs and those from the skin. We will consider first,—respiration. Experiments show that a person inhales, when breathing quietly, from 20 to 34 cubic inches; when walking respectively 1, 2, 3 or 4 miles per hour, about 52, 60, 75, or 91 cubic inches; and in working a tread-mill, 107 cubic inches of air—per respiration. The amount of carbonic acid exhaled is equal in bulk to the oxygen consumed, and is about 4.5 per cent. of the air inhaled. Estimating that a laborer in the caisson consumes as much air as a person walking 3 miles per hour, we have with 18 respirations per minute, the air breathed by 70 men, $75 \times 70 \times 18 = 94{,}500$ cubic inches. Of this 4.5 per cent. or 4252 cubic inches is carbonic acid, exhaled and thrown off by the lungs.

The watery vapor expelled by the system at the pores, is stated to be, by weight, from 1.5 to 2 times the amount of carbonic acid exhaled, and contains carbonic acid, urea, and albuminous substances in a state of decompostion. In the air of the caisson, owing to its nearly constant state of saturation with watery vapor, this vapor would no doubt be given off in the form of "sensible perspiration." In the absence of any data, allow that it contaminates one-quarter as much air as the breath, and we get carbonic acid and the like from the skin equal to $\frac{4252}{4} = 1062$ cubic inches.

We come next to the carbonic acid produced by combustion. Carburetted hydrogen consumes in burning, twice its bulk of oxygen; and generates its own bulk of carbonic acid. In other words, a cubic foot consumes 10 cubic feet of air, and produces a cubic foot of carbonic acid. As we were burning about 5 feet of gas per minute, this gave carbonic acid from the lights equal to 8640 cubic inches, and from all sources as follows:

Breath	4252	cubic inches
Skin	1062	" "
Gas	8640	" "
Total	13954	" "

or 8.08 cubic feet.

The next thing to determine, was the per cent. of impurity allowable in the air. Pure air contains by volume, in round numbers, .8 nitrogen and .2 oxygen. There is present, also, a small amount of carbonic acid variously estimated from .1 to .04 of one per cent. "10 per cent. of carbonic acid causes instant death; 5 to 6 per cent. is dangerous to life, and 3 per cent. causes a candle to cease burning." "It is decidedly prejudicial to breathe for any length of time, air containing one per cent. The air in the latter case becomes soporific, depressing, and altogether injurious." Writers on ventilation recommend that the amount present should never be greater than .2 of one per cent. Taking, as is given by some writers, 3.5 to 5 cubic feet as the amount of air actually vitiated by breathing alone, per man per minute, breathing at the very moderate rate of 30 to 40 cubic inches per inspiration with 18 inspirations per minute, would give .33 to .5 of one per cent. of vitiation. As a man laboring hard would certainly breathe more air than this, the vitiation would probably reach one per cent., which was too large to be permitted.

I assumed therefore, a purity of .33 to .5 of one per cent as being desirable, giving at once from $\frac{8.08}{.0033}$=2424 cubic feet to $\frac{8.08}{.005}$=1616 cubic feet per minute as the necessary supply.

Our compressors throw 3 cubic feet of air per revolution, and make 60 to 80 revolutions: hence it was necessary to run at least 8 of them to keep the air wholesome. As we had previously run from 4 to 6 only, orders were given to ventilate the chambers by blowing off at frequent intervals, and no trouble was experienced afterward. The fact that more air was needed for health than was necessary to supply the waste by leaks, was one of the leading reasons for its use in removing the material excavated.

Subsequent to this investigation, when 10 to 12 compressors were in constant use, the air of the caisson was tested for carbonic acid, and about one-third of one per cent. found to be present, thus verifying the previous results.

HEALTH.*

There has been so much adverse criticism by the public press on pneumatic foundations, on account of the fatality attending them, this paper would hardly be complete without giving the result of our experience in this regard.

On the Brooklyn side the men worked 8 hours, in 2 shifts of 4 hours each, down to the full depth of 44.5 feet, without injury. On the New York side the time was reduced to 7.5 hours, at 45 feet; to 7 hours at 50 feet; to 6 hours at 60 feet; to 5.5 hours at 70 feet; to 5 hours at 71 feet; to 4.5 hours at 76 feet; and to 4 hours at 77 feet.

The first fatal case, which was considered fairly attributable to the compressed air, took place at the depth of 75 feet, from congestion of the lungs. The man was recently employed, of very full habit, and had worked but one shift of 2.5 hours. He felt well after coming up, but died about an hour afterward. When examined two days before, his lungs were sound. What subsequent exposure he may have had, we do not know. One man died previous to this of cerebro-spinal meningitis, and one afterwards of Bright's disease of the kidneys; neither caused (although possibly hastened) by the work. One other new man died of congestion of the spine, due no doubt, to the work. He also was fleshy, and it is thought, of intemperate habits. Another fatal case was reported, the man having died, it is said at home; the case is yet to be examined. There were perhaps a dozen cases of paralysis of considerable severity, but all recovered in from three days to three weeks. At from 50 feet depth to the end, severe pains in the legs and arms (called by the workmen the "Grecian Bend") were frequent, but

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* RULES FOR THE WORKMEN IN THE CAISSON.

1st.—Never enter the Caisson with an empty stomach.

2d.—Use, as far as possible, a meat diet, and take warm coffee freely.

3d.—Always put on extra clothing on coming out, and avoid exposure to cold.

4th.—Exercise as little as may be during the first hour after coming out, and lie down if possible.

5th.—Use intoxicating liquors sparingly; better none at all. It is dangerous to enter the Caisson after drinking intoxicating liquors.

6th.—Take at least eight hours' sleep every night.

7th.—See that the bowels are open every day.

8th.—Never enter the Caisson if at all sick.

9th.—Report at once to the office all cases of illness, even if they occur after returning home.

did not last long. Most of the cases were found due to disregard of some of the rules of health, which were furnished every workingman in printed form. The four rules certain to cause trouble, when violated, were the ones relating to rest, sleep, eating, and the state of the bowels.

At an early stage of the work on the New York side, a physician was employed to spend from one to four hours per day at the pier. His visits were always made in the afternoon, as cases of sickness on the early shifts were rare. At other times the engineer on duty, or some person under his direction, attended to the men.

The remedies employed were simple: prompt use of ergot and morphine would generally alleviate pains in the limbs: stimulus together with Jamaica ginger were given for epigastric pains. Where vomiting set in, and was persistent, the physician was sent for, as paralysis frequently followed.*

Coffee was always served to the men, immediately after coming out of the caisson. Bunks were provided also, in which all who wished, could rest. It would have been well perhaps, had the use of these been compulsory, as it was by no means general.

One important conclusion from the records kept of cases, is this; that the greater number of those who have retained their health throughout are wiry, somewhat spare men; while most of the sick, and all who died, were fleshy men, of full or large size.

Dr. A. H. Smith, the physician in charge, intends to publish his observations in full; and will be glad to receive any information regarding similar work, that he may get all available facts concerning the peculiar symptoms, sickness and treatment.

SAND PUMPS.

I have been requested to give a very brief statement of experiences in well boring;† the only thing new in connection with which, as practiced on the work, was the method of sand-pumping.

No difficulty was found in removing by means of the ordinary sand-pump, the sand and mud from the tube after driving, but the pebbles

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* It was found that cold water poured on the spine, or the part affected, and followed by friction was of great benefit in all the cases.

†As a note to this paper, and not read before the Convention.

being of all sizes and exceedingly hard, whenever a layer of them was reached, the progress was vexatiously slow.

The drive pipe used was 6 inches in diameter. The sand-pump was 4.5 inches diameter and 5 feet long, with a simple valve at the bottom; and when clean pebbles only were in the pipe, would frequently come up without anything in it. To overcome the difficulty I made an ordinary conical plunger of leather fitting loosely in the pump and attached to an iron plunger bar, heavy enough to carry the apparatus and lifting rope promptly down. The pump and plunger were lowered together, and then letting the pump rest on the bottom, the plunger was worked with a sharp, short stroke, of course not far enough to lift it from the pump. At the very first trial of this clumsy apparatus, the pump came up more than half full of pebbles; and the same success continued to follow its use.

This result could be reached more elegantly by having two tubes, each with a valve ; a foot valve at the lower end of the lower one, and suction valve at the upper end of the upper one. These tubes could be arranged, the upper to slide over the lower, or over an intermediate one to which the lower was attached, and all to lift together when the upper is at the end of its upward stroke; a lifting rod heavy enough to sink the rope promptly should be attached to the upper tube. There can be no question that such a sand pump would work well and save an immense amount of time if the valves were well arranged. It would give a positive lift, directly into the pump, in place of an accidental deposition as an effect consequent upon agitating the water by churning the pump up and down. This device was not tried, for the reason that it was not made until the borings were nearly completed.

Mr. J. Dutton Steele : There is one subject I would like to have a little light upon—it is this : the depth to which caissons can be sunk safely by using compressed air. During the consideration of the plan for the East River Bridge, it was estimated that the caisson should be put down as low as 105 feet; and it was a subject of grave consideration with Mr. John A. Roebling and the gentlemen in consultation with him, whether that depth could be attained with safety to the workmen ; and if of delicate physical organization, they would not be destroyed by the pressure induced. That caisson has gone down 80 feet, and the result is as feared—the pressure was hazardous to persons delicately organized. It is stated that a caisson at St. Louis has been

put down 110 feet, which indicates that 30 feet deeper can be as safely sunk there, as that which has been reached in New York.

Was this depth of 110 feet occasioned by a temporary rising of the water, or was it a steady pressure? An accurate knowledge of the facts ascertained is important, since, as will be seen, upon it depends our knowledge of the depth to which we can safely go by compressed air.

MR. KATTE: As my work is entirely upon the superstructure of the St. Louis bridge, and my observations of the foundations were only made at rare intervals when I was stationed in St. Louis, I can only state, that as far as my recollection goes, the greatest air pressure the men were subjected to in sinking the eastern abutment,—which is the deepest,—was something over 50 pounds. I am aware there was far less difficulty experienced from the effect of the air upon the men in the lowest foundations than in some of the others. The experience gained in the mode of treating the men in the former foundations seems to have been entirely efficacious. I forget the exact figures as to depth, but I think it was about 110 feet, with a pressure of about 50 pounds.

MR. COLLINGWOOD: In the New York caisson, East River Bridge, there has been a moderate amount of sickness, and one or two deaths. The St. Louis caisson was sunk 30 feet deeper, with similar results, indicating that still greater depths may possibly be reached.

The law practically admitted in adjusting the hours of labor in the caissons, both at St. Louis and New York, seems to be as follows: taking it for granted that 12 hours is the extreme time a man can labor without detriment to health, in an ordinary atmosphere; then, with a pressure of 2 atmospheres, or 15 pounds additional pressure, he can labor about one-half that time; with 3 atmospheres, about one-third of that time; and with 4 atmospheres, about one-fourth of that time. In other words, the time is inversely as the pressures.

The men will not be injured, provided a proper time is given them to rest. In saying this, it does nor follow that men should work continuously, the number of hours thus indicated; experience shows the contrary; and the time of work has invariably been divided into two portions, with a reasonable interval between.

It seems that there is a possibility of accommodating the men, in this way to the pressure; the only difficulty is, the return to the normal pressure. It is clearly the duty of every member of the profession who has anything to do with such work, to make public, as far as possible, the remedies used under the circumstances.

Mr. McAlpine: I think Mr. Collingwood has struck out an idea. The compressed air acts as a stimulant; pretty much like those commonly used, and is ultimately exhausting. I fancy that is precisely the explanation of its effects. With one atmosphere a man may work 12 hours; and with an increased pressure the number of hours must be diminished. I myself have been stimulated under this pressure, almost as if I had taken laughing gas. With workmen whom I employed during a period of 8 months under a pressure from one to five atmospheres, there was nothing that caused the slightest inconvenience, except when the pressure was put upon them too suddenly. By gradually increasing the pressure no unpleasant effects were present or visible.

XLVII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

A DESCRIPTION OF THE PROPOSED PLAN FOR ERECTING THE SUPERSTRUCTURE OF THE ILLINOIS AND ST. LOUIS BRIDGE.

A paper read by WALTER KATTE, Civil Engineer,* Member of the Society.

The problem of how to sustain and maintain in position the arch ribs of the Illinois and St. Louis Bridge, during the progress of their erection, is one regarded by all engaged upon the construction of the truly great work as second only, if not equal, in importance and interest to the one completed in so signally successful a manner,—the sinking of the piers and abutments to their foundations on the bed rock.

After the most careful study of all the conditions under which this work has to be performed, a plan has been adopted, the prominent features of which, divested as much as possible of the minutiæ of detail, it is the object of this paper to describe.

The most prominent novelty in this plan consists in the absence of all scaffolding or trestling standing in the river, excepting only for a very short distance immediately adjacent to the piers and abutments, substituting therefor a suspending system, from above; also in using

* Engineer of the contractors, the Keystone Bridge Co.

the inherent stiffness of the arch ribs themselves as cantilevers, (aided, if found necessary, by temporary "guys" from the piers and abutments,) to support the derricks and stages from which to project forward the successive sections of the ribs.

The reasons which led to this radical departure from all the usual types of "falseworks" are found in a consideration of the unstable and insecure foundation offered by the shifting sand of the river bed, which—liable to deep and sudden scour at every flood—precludes the use of any system of trestling or piling, unless carried down to the bed rock at an enormous outlay, not only in expensive material and workmanship actually consumed in the works themselves, but also for the subsequent removal of the same, and the loss of large quantities of material which could not be recovered except at unremunerative expense; also in the cost and maintenance of an effective system of protection works to guard the "falseworks" proper from the danger of colliding rafts and river craft, all of which would have to be placed in position and again removed, three successive times, in the deep and rapid current of the Mississippi river; the exigencies of the crowded river traffic in this, the very centre of the harbor of St. Louis, not permitting the closing of more than one span at a time.

Under these considerations, it was suggested by Colonel Henry Flad, Principal Assistant Engineer of the Illinois and St. Louis Bridge Company, to abandon all idea of invading the river with any system of "falseworks" whatever, and to adopt the principle of suspension from above in some form. He elaborated his ideas to some extent, and made preliminary calculations, which so entirely demonstrated the practicability of his views, that I was instructed by the managers of the Keystone Bridge Company to work out the complete details of the proposed plan, in doing which, I have received most valuable assistance from the mature judgment and advice of Captain Eads, Engineer of the Illinois and St. Louis Bridge Company, Colonel Henry Flad, and of Messrs. J. H. Linville, President, and J. S. Piper, General Manager, of the Keystone Bridge Company.

Each arch rib is in the form of a polygonal segment, consisting of an upper and lower member, spaced 12 feet apart from centre to centre, each member being a series of steel tubes of 18 inches diameter and about 12 feet in length. The number of segments composing each rib are as follows: in the side spans,—upper members, 42, lower

members, 43; in the centre span,—upper members, 44, and lower members 45. *

Accurate calculation demonstrates the ability of the ribs, when divested of the moving load and weight of roadways, platforms, and their supports, to sustain themselves as cantilevers projected from their skewbacks, for a distance equal to about one-fourth the entire span, (or to joints Nos. 12 and 30); the skewbacks being immovably secured to the masonry by immense bolts of steel and iron passing entirely through the piers, from skewback to skewback, and into the abutments far enough to secure adequate anchorage, by means of cast iron washer plates built into the masonry. † This being the case, it will readily be perceived that if joints Nos. 12 and 30 are sustained by cables passing over a temporary tower, reared above the piers and abutments to such a height as will afford an economic angle, and with strains exerted in them, to equilibriate the gravity of the semi-girders with their extraneous loads of staging, hoisting, and other erecting machinery sustained by them, and also elevate joints Nos. 12 and 30 to their proper horizontal position by taking up the deflections, then the remaining quarter spans may, by repeating the process, be projected forward until they meet in the centre of the span; the deflections of these last quarters of the spans must be taken up by a secondary cable system, carried from the centre sections over masts erected at joints Nos. 12 and 30, and back to an anchorage to the ribs themselves near their skewbacks.

By careful calculation it is found that a strain of 216 tons in the primary, and 108 tons in the secondary cables, will fulfill the foregoing conditions, and sustain joints Nos. 12, 22, 23 and 30 at their normal heights, the temperature being 60° Fah. The two semi-ribs having thus been projected, leaving only the closing section to be inserted to complete the arch, it will be found that their ends are too near together by an amount equal to the difference between the compression in the whole arch produced by its weight and superincumbent load, and that produced by the artificial strains in the cables, the former being 3.4 inches, and the latter 0.464 inches in the upper tubes and 0.257 inches in the lower ones, leaving 1.236 inches in the upper and 1.44 inches in the lower tubes, of compression or contraction still

* Mr. Katte said: The middle span is 515 feet long, from centre to centre of skewback, and the end spans 497 feet long.

† Mr. Katte said: The bolts vary from 22 to 38 feet long.

to be accomplished in each semi-rib by some other means before the closing section can be inserted; the suggested modes of effecting this will be hereafter described.

Economical considerations induced the decision to project only the two middle ribs to a "close,"—in the centre of the spans by means of the cables, stopping the erection of the two outside ones at joints Nos. 12 and 30 until the completion of the two middle ones; these will afford a perfected arch of two ribs 12 feet apart, by means of which, from cantilever stages projecting laterally from them, the remaining sections of the outside ribs can conveniently be put in place.*

The two outside ribs, when projected to joints Nos. 12 and 30, will be sustained by sub-cable systems attached at joints Nos. 6 and 36, to take up their deflections, so that their weight, acting through the diagonal tension rods, may not be imposed upon the middle or main cables; it is found that a strain of 130 tons on these cables will elevate joints Nos. 12 and 30 to their proper height; at the abutments they will be carried back to an artificial anchorage obtained by loading down a stage with railroad iron.

The derricks, and working stages suspended from them, will travel forward section by section as the work advances; they will be moved by rack and pinion gearing, and travel on movable tracks, mounted in successive sections, on the top of each rib, and secured to the upper tube by clamp-bands. There will be a crab to each of the four ribs, with booms projecting beyond the tubes, from which the machinery will be hoisted from barges anchored immediately below. The four derricks will be all connected together laterally by ties and braces, so as to act as one frame, mutually stiffening and supporting each other. †

I now proceed to describe the cable systems and accessories under the following heads:

I. The Timber Towers and Guide Trestles.

II. The Hydraulic Rams and their Balance Gauges.

III. The Cable Chains and their Anchorages.

IV. The Cables for Side Ribs.

V. The Secondary Cable System.

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* Mr. Katte said: We will then have the completed arch 12 feet wide over the whole span, and by stages from that the outside rib can be added.

† Mr. Katte said: One of these derricks must be attached to each tube; each is to be clamped by screw clamps passing entirely around the tube, so

I. The Timber Towers and Guide Trestles.—The towers for each pier and abutment consist of two pyramids of timber about 50 feet in height, formed with a central post or mast of four yellow pine whole sticks, 12 inches by 12 inches, strongly framed together with oak keys and iron bolts in both directions, thus combining them into a solid post 24 inches square, footed into a suitable casting resting on cross sills, which are of oak timber 18 inches by 24 inches; in order to preserve their full strength, these sills, at their crossing, are not "halved" together, but simply crossed and locked with an inch "lock joint" and through bolts, the difference in the level of their under surfaces being made up of oak blocking, bolted to the under side of the upper sill; from the four extremities of the cruciform sills, four braces of yellow pine, 12 inches by 12 inches, are carried up nearly to the top of the centre mast, inclining from it at an angle of 6°; at their upper ends they are securely framed and bolted to the four sides of the centre post, with intermediate ties and stout bracing between them and the mast, at intervals between the top and sills; the two pyramids are also connected together laterally throughout their whole length by a strong system of struts and ties, thus mutually supporting and stiffening each other. The top of each mast is surmounted with a suitable casting, forming the apex joint of the cables.

To stay or "guy" these towers in a lateral or "up and down stream" direction, two guide trestles are employed, standing 12 feet apart, and firmly anchored to the masonry. These trestles have sills 60 feet, and caps 20 feet—long, and stand 48 feet in height; they are framed of pine timber, 12 inches by 12 inches, with three vertical posts 7 feet 9 inches apart from centre to centre, and with two leaning posts or braces inclining to the verticals, at an angle of 24°. The posts and braces are strongly secured to the caps and sills by heavy iron straps and bolts; they are also stiffened transversely by a strong system of clamps and braces; at the tops these two trestles are connected by horizontal tie beams and braces, so disposed as to leave rectangular openings or "slots" through which the upper ends of the masts are free to move up and down, and in the direction of the axis of the bridge, and to resist any tendency to tranverse movement. *

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that it will stand firmly upon the work itself. As one section is built, the derricks are to be moved forward.

* Mr. Katte said; In the middle section there is a tendency to sway, caused by the different expansion of the cables, so that we can move it up and

The cruciform sills of the towers stand on the "cross heads" of two hydraulic jacks or rams, which are located directly under the centre of the masts, the office of which is to maintain a constancy of strain in the cables, under changes induced by variation ot temperature. Thus the towers standing on their two rams are free to move up and down, as the plungers move up and down in the cylinders, to compensate for the contraction or expansion in the cables and tubes induced by thermal variation. To guard against accident which might arise from the possible failure or derangement of the rams or gauges, a properly arranged "blocking" will be kept constantly under the tower sills, with gum cushions or some other suitable device interposed, so as not to interfere with the free action of the rams.

II. The Hydraulic Rams and their Balance Gauges.

The cylinders of the rams are 13 inches in diameter and 30 inches in height, the metal is 7 inches thick, and there is 12 inches length of stroke provided for, though but 6½ inches is actually needed; the plungers are solid, 13 inches in diameter, and the packing is of leather, cup shaped, in recess of cylinders near the top; the area of the plungers being 132.75 square inches, and working pressure 2560 pounds per square inch, the working power of each ram is 170 tons; which is the calculated work required of them, being the vertical component of the cable strains plus the weight of towers, &c; ample provision is made for additional strain that may be required, by the rams and all their gauges and connections being tested to 5000 pounds per square inch, before being put into service.

The crossheads are disconnected from the plungers, and rest upon them, in a planed rocking joint. Should any accident happen to the rams, requiring their temporary withdrawal or replacement, it can readily be done by introducing four smaller jacks or rams under the leaning brace-posts of the towers, by which means the main ram may be entirely released.

The gauges by which the ram pressures are to be governed and measured will be long wrought iron hydraulic tubes, 2.284 inches in diameter, with plungers weighted with movable "pea weights," working in them, having a stroke of 7.5 feet. The object of the long stroke is to make these gauges *automatic* governors of the strains, within a

down, and also move it slightly one way and another at the top. The different expansion in the cables induces a very slight motion under the changes of atmosphere.

certain limited range of thermal variation, thus determined: By minute calculation of the extension and compression induced by the strains in the material of the cables and tubes, together with the effects of a range of 80° each way from a normal of 60° Fah.—or from 20° below zero to 140° above—in producing expansion and contraction, we find that the apex of the cables will change its position vertically 6.5 inches in maintaining an unvarying strain within that range of thermal variation; this then, is the total stroke of the ram, which being 13 inches in diameter gives 862.5 cubic inches of the fluid in the ram, corresponding to 140° range of temperature. Now the gauge tube being 2.284 inches in diameter, 7.5 feet stroke will move 319.25 cubic inches of fluid; and as one gauge governs two rams—one half that amount, or 159.625 cubic inches, being 18.5 per cent. of 862.5 cubic inches,—it follows that the stroke of the gauge controls that percentage of thermal range of the ram, or 29.6°. Hence within that range the gauge is "*self-acting*" in maintaining constancy of pressure. Beyond that range, the "pea," by rising or falling to the extremity of its stroke, gives notice to the person in charge of the rams that he must let off, or force in fluid, as the case may be, to again restore the "pea" to its midstroke position. Provision is made for gradually increasing the strains as the work advances, by the addition of successive weights to the "pea." When the cables are first attached at joints No. 12 and 30, the strain required is only 65.5 tons, obtained by a weight of 3,200 pounds on the gauge. This is the starting weight; after that, for each section of tube added, a weight of 100 pounds is added to the gauge, keeping, however, the strain a little in excess of that actually required to balance the weight; to give rigidity against suddenly imposed or relieved loads on the working stages, and to still further guard against vibratory action in the rams and gauges that might arise from these causes, it is proposed to interpose a diaphragm with only a small hole in it, at some point in the connections between the rams and the gauges, which while allowing the pressure gradually to equalize itself, would effectually guard the gauges from sharing in any sudden vibrations in the rams.

III. The Cable Chains and their Anchorages.

The Cables will consist of wrought iron eye bar links, each 27 feet 6 inches long and 1 inch thick; alternately seven bars 6 inches wide and six bars 7 inches wide, thus giving a cross sectional area of 42 square inches. The heads are upset; the working strain allowed on the links is calculated at about 5 tons per square inch.

The connecting pins are 3.5 inches in diameter. At each end of each cable a set of adjusting links is introduced of 2.25 inches by 2.75 inches, with screws of equal strength and long "right and left" sleeve-nuts or swivels—for the purpose of taking up slack and bringing all the strands as nearly as possible to an equal tension before the ram pressure is applied. They are attached to the upper tubes of the arch ribs by heavy wrought iron forged clamps, each completely encircling the tube, just in front of the joint-coupling, against which it rests.

On the St. Louis side, the anchorage is obtained by sinking vertical wells into the solid rock, to a depth of from 25 to 30 feet, and then undercutting the rock to form chambers to receive heavy castings, to which the links are secured by eye-bolts. On the Illinois side, where the surface formation is entirely of sand, an artificial anchorage has to be provided—which will be done by making an excavation and driving within it four rows of sheet piling,—each pile 12 inches by 12 inches,—in a direction at right angles to the plane of the cables, and of sufficient breadth to engage a sufficient body of sand to amply resist the pull of the cables.

Behind the sheet piling a beam is placed, built of oak timber, 12 inches by 12 inches, keyed and bolted together in both directions, so as to form a solid beam 4 feet square, and distribute the strain over the whole breadth of the sheet piling. To this beam the cables are secured by oak crossheads and wrought iron keys, in a manner to distribute the strain over the whole breadth of the beam. After the cable attachments are made, the whole excavation will be filled up and rammed, leaving the beam buried behind its sheet piling some 30 feet below the surface.

IV. The Cables for Side Ribs.

As before mentioned, the side ribs will be sustained by another system of cables, in order that by their deflection they may not throw additional strain on the main cables, through the diagonal bracing rods. These cables will have a working strain of 130 tons, and a sectional area of 26 square inches; they will be the same in general form and arrangement as the main cables, being alternately three bars 7 inches by 1.25 inches, and four bars 6.5 inches by 1 inch. They will be provided with screw-adjusting links at each end, and attached to the tubes with clamps in the same manner as the cables; they will be carried over the piers and abutments on very simple "saddle blocks." No hydraulic rams will be used with these cables,

their office being simply to hold the outside ribs at the same level as the middle ones, which can be done with screw adjusting links.

A convenient anchorage for the abutment end of these cables is obtained behind the centre wall of the abutments, where an offset or step occurs, against which the edge of a platform can rest, with a mass of solid masonry in front of it, amply sufficient to resist the "horizontal" component of the cable strain, while the "vertical" will be resisted by loading the platform down with railroad iron.

IV. The Secondary Cable System.

The details of the secondary cable system, required for elevating the middle arch ribs at their centre joints, have not as yet been fully determined upon. In general, however, it will consist of masts erected at joints Nos. 12 and 30, (where the main cables are attached), with cables of link bars, carried from the head of them to the joints next back of centre joint, and with back-stays attached to the tubes near the skewbacks. The strain required on this cable is 110 tons; it is as yet undecided whether this will be obtained by hydraulic power or by screws.

When the middle ribs have, by the foregoing described process, been projected to the centre, so that nothing remains but to fit in the closing section of tubes to complete the arch, a difficulty arises, how to surmount which is still the subject of much study.

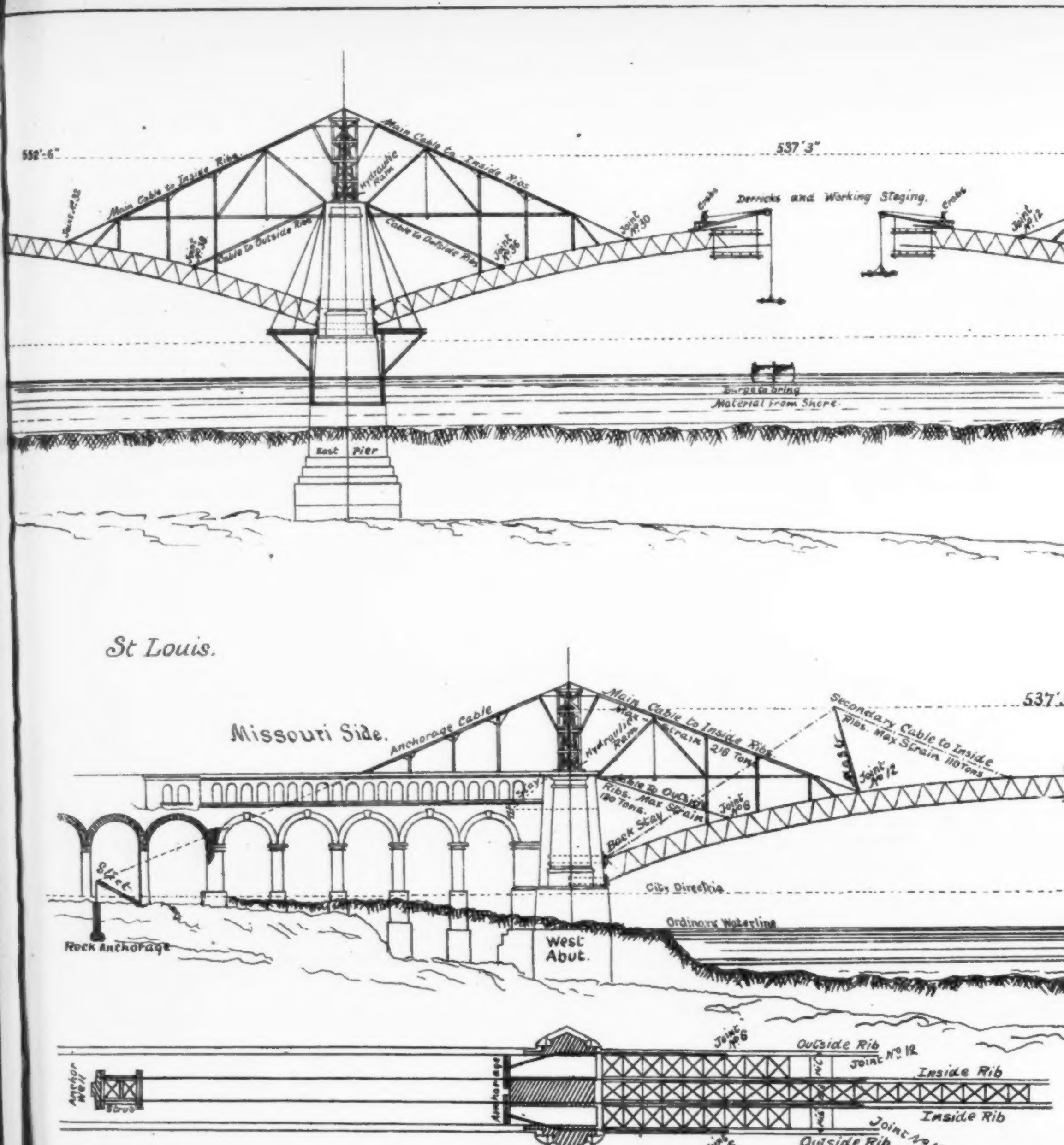
As before mentioned, the length of the unloaded arch rib at the normal temperature of 60° Fah., is 3.4 inches longer than it will be when complete and loaded; hence the space left for the closing section at a temperature of 60° will be that amount, (3.4 inches) short for the tube. The problem then is how to produce such an enlargement of this space that the closing section may be inserted; a portion of this is obtained by the compression induced by the cable strains—equal on both the semi-ribs to .464 inches in the upper tubes, and .257 inches in the lower tubes,—leaving 2.472 inches in the upper, and 2.886 inches in the lower tube, still to be obtained in some other way.

Should the closing of the arches happen in mid-winter, or when the temperature ranges the lowest, the difficulty is materially lessened in this way:—the entire length of the arch ribs being 6.357 inches, a fall of temperature of 68° below the normal 60° Fah. would produce a contraction of about 2.88 inches (at the rate of .001 per 150°);—should, therefore, the mercury range from 8° to 10° below zero, the centre section could be entered without difficulty. Should the temper-

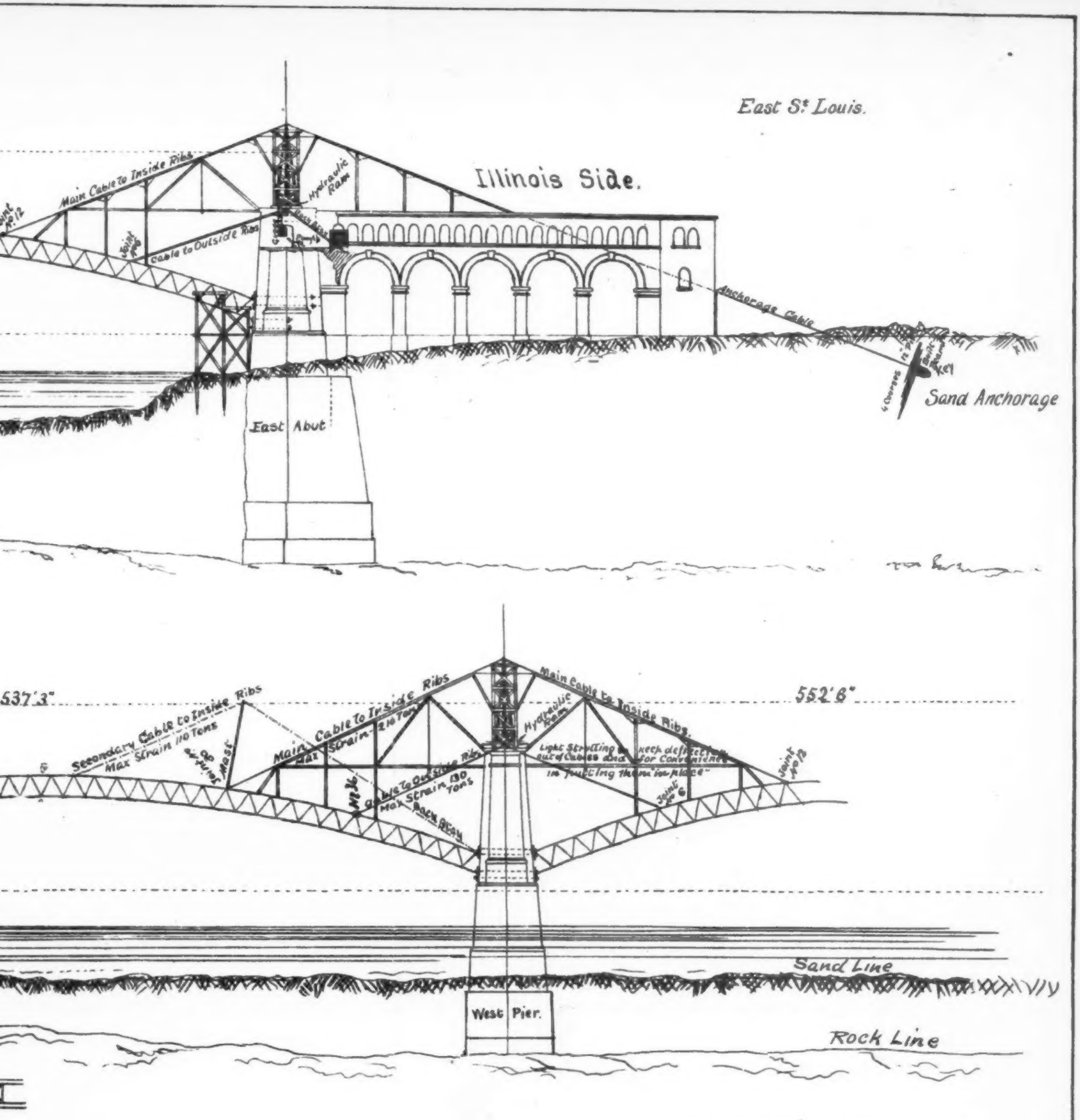
ature not range as low as that, at the time it is necessary the work should be done, it has been suggested to produce the required reduction of temperature in some artificial manner, such, for instance, as passing a stream of cooled compressed air through the tubes, the air to be,—by passing through an ice-box, for instance,—entirely deprived of the latent caloric evolved by the compression, and then being forced through the tubes of the arch ribs it will, while expanding in its passage, abstract caloric from the tubes, they being protected from the external atmosphere by wrappings of non-conducting material. In this way it is believed a reduction of temperature may be produced, sufficient to give the desired contraction.

It is not possible to employ direct pressure to effect this object, owing to the great weight of the machinery and appliances required to produce it, which would throw inadmissible additional strains upon the tubes and supporting cables. It is possible that the ends of the semi-ribs may be sprung up sufficiently in the centre to allow the centre section to be entered; but this plan has not yet been fully investigated.

The order of erection will be as follows: starting simultaneously from the western abutment, and from both sides of the west pier, the four ribs will be completed to joints Nos. 12 and 30 in the western and centre spans, and the two middle ribs completed in the western span. The towers and cables will then be transferred from the west abutment to the east pier, from which the four ribs will be carried out on both sides to joints Nos. 12 and 30, and the middle ribs closed in the centre span. The towers and cables will then be transferred from the west pier to the east abutment, when the four ribs will be built to joint No. 30, and the middle ones to the centre of the east span. In the meantime, the addition of the outside ribs, between joints Nos. 12 and 30, will be going on simultaneously, upon stages "outrigged" from the completed middle ribs.



550'-6"
537'3"
Main Cable to Inside Ribs
Main Cable to Inside Ribs
Hydraulic Ram
Cable to Outside Ribs
Cable to Outside Ribs
Derricks and Working Staging.
Crabs
Barge to bring Material from Shore.
East Pier
St Louis.
Missouri Side.
Anchorage Cable
Main Cable to Inside Ribs
Max Strain 216 Tons
Hydraulic Ram
Secondary Cable to Inside Ribs. Max Strain 110 Tons
Cable to Outside Ribs. Max Strain 180 Tons.
Back Stay
Joint No 6
Joint No 12
537'
City Directrix
Ordinary Waterline
Rock Anchorage
West Abut.
Anchor Well
Strut
Anchorage
Outside Rib
Joint No 12
Inside Rib
Inside Rib
Joint No 12
Outside Rib
Joint No 6
Joint No 6



ROCKWOOD'S FAC SIMILE PROCESS. N.Y.

XLVIII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

EXPERIMENTAL TESTS OF BUILDING STONES.

A paper presented by ROBERT G. HATFIELD, Architect, Member of the Society.

TRANSVERSE STRENTH.

In the Journal of the Franklin Institute, March, 1858, are recorded the results of some experiments which I made to test the transverse strength of stones in use as building material.

These experiments were made upon a testing machine operated by hydraulic pressure, and although care was taken in the construction of the machine and in the selection of the fluid used to operate it, so as to avoid friction, yet the results were necessarily unsatisfactory: at the best they were only tolerable approximations. They were sufficiently uncertain to make it desirable to procure a machine for testing, free from the friction due to hydraulic engines.

A determination to have such a machine has long been cherished, but the ceaseless round of professional engagements has deferred for years the accomplishment of the desire.

During the past winter an occasion of more than ordinary urgency compelled me to give my attention to the subject, and a machine was planned and built. It is a platform scale set in a table, and so arranged with hand wheel and gearing as to produce a *pressure* upon the platform. This pressure is transmitted in the usual manner by levers on

knife edges to the scale beam. By an ingenious contrivance suggested by my son, the poise upon the scale beam, instead of being moved by hand as the pressure is applied, is made to travel by a clock movement, stopping automatically when reaching the point on the beam which represents the pressure upon the platform.

By a careful handling of the wheel and gearing the pressure is increased no faster than the poise travels. At the moment of rupture, the poise is arrested and indicates truly the highest pressure attained. By loading the platform with weights, the action of the poise movement has been tested and found to be correct to within two pounds.

Table I exhibits the results of some of the experiments made with this machine.

The specimens numbered 11, 13 and 41 broke at an unknown pressure. The failure to obtain the strength in these cases was owing to the fact that generally in making experiments, the scale beam is loaded at the start with a weight supposed to be far within the strength of the piece tested, in order to economise time in making the experiment; but, in these cases referred to, the said weight proved to be too great. Generally, such a premature rupture in a specimen is owing to accidental weakness. The stone at Nos. 23, 24 and 25 is a new sand stone of a beautiful greenish tint, lately introduced at Cincinnati, and is there known by the name of Constitution Stone.

The specimens experimented on were kindly contributed for the purpose by the following gentlemen, viz: Mr. Edwin Anderson, Architect, of Cincinnati, Ohio, furnished specimens numbered 23, 24 and 25; Messrs. A. Hall & Sons, of Perth Amboy, New Jersey, furnished specimens numbered 47, 48, 49 and 50; Mr. S. W. Brainard, of Brooklyn, E. D., New York, furnished specimens numbered 4 to 11 inclusive; and the following gentlemen of this city furnished specimens as follows: Messrs. Sinclair & Milne, numbers 1, 2, 3, 14 to 22 and 28, 29 and 31; Mr. R. L. Darragh, numbers 33 to 46; and Messrs. Fisher & Bird, numbers 12 and 13. I desire here also to express my acknowledgements to Messrs. Fisher & Bird for their kindness in preparing several of the specimens for the tests, by sawing them into pieces of appropriate size, at their extensive marble works.

In the formula, $S = \frac{lw}{bd^2}$, l is in feet, and b and d are in inches; while w is in pounds. Hence S is the weight in pounds required to break a bar one inch square, and one foot long in the clear between the

bearings. In the fourth column of the table the lengths of the specimens tested are stated in inches, the breadths and depths are also in inches, and the breaking weights in pounds.

DURABILITY.

Among the causes of decay in stone, that of freezing may be named, perhaps, as one of the greatest. The effect of freezing will be in proportion to the power of absorption, or to the porosity of the stone ; while the resistance to freezing will be in proportion to the tenacity with which the particles adhere to each other. The tenacity is shown in experiments made to test the tensile strength of the material. It is also shown very fairly in experiments on transverse strength ; for in this material the resistance to a tensile strain is so small in comparison with its resistance to compression, that, in a beam supported at each end and loaded in the middle the neutral plane rises nearly to the upper edge of the beam ; or, the rupture occurs by the yielding of the material to a strain almost wholly tensile. In Table I, therefore, we have a comparative view of the tenacity of the various stones named ; and if to this we add a knowledge of the comparative porosity we are then possessed of the two required elements in the formula for resistance to disintegration by freezing. In Table II, may be found the porosity of some of the material named in Table I.

For the experiments required to obtain the porosity, &c., as in Table II, I am indebted to the kindness of my friend Professor C. F. Chandler, Ph. D., of the New York School of Mines, who has manifested a lively interest in the tests made upon the specimens furnished.

In obtaining the porosity, the specimens were immersed in water from which the pressure of the atmosphere was removed, and then permitted to return, thus securing most thorough saturation. The specific gravity given is not that of the solid particles alone, but of the solids and voids combined. The specific gravity of the solid was obtained from actual weighing. The specific gravity of the Table was computed from this and from the porosity.

In regard to the sulphate of soda test, or artificial freezing, the specimens were immersed in a saturated solution for one day and night, and the apparent action is noted in a scale of from 1 to 12, the larger numbers indicating the greater action of the soda. Other speciments of the same stones were surrounded with small pieces of ice and left for the same length of time. These when examined

seemed to be affected nearly to the same degree as those submitted to the sulphate of soda test.

The experiments in Table II may be of use in showing the porosity of the stones named and in comparing the density with the weight and volume of the material in each case.

In order to make a more decided test than was possible with sulplate of soda, I exposed a few specimens to the continued action of frost during the past winter. For this purpose I placed each specimen in a small covered vessel filled with water, upon the sill of a window having a southern aspect. The vessels containing the stone were placed upon the window sill on the 23d of December, 1871, and there remained day and night, until the month of April, 1872. During this time the water in the vessel was alternately frozen and thawed many times. During some of the coldest days the water, owing to the effect of the sun, was thawed at noon and again frozen before sundown. At the end of the winter the water was carefully syphoned from each vessel and the contents dried by artificial heat. The disintegrated particles were carefully gathered and weighed, and the weight in French grams divided by the superficial area exposed to freezing :—the quotient is shown in the fifth column of Table III. If the amount of disintegration experienced by the specimens be taken as the work of one year, and this an average year ; then the durability of each stone, in years, is expressed in the sixth column of Table III.

Hoping that the few facts exhibited in this paper may contribute in some slight degree to the stock of knowledge of my fellow members, I submit them for their consideration without any further attempt at application.

TABLE I.

No.	Name.	Location.	Length. l.	Breadth. b.	Depth. d.	Breaking Weight. w.	$S = \frac{lw}{bd^2}$.
1	Grewacke.	North River, blue.	15.	2.92	1.50	1318.	250.76
2	"	" "	15.	3.04	1.45	1191.	232.92
3	"	" "	15.	3.07	1.43	1141.	227.19
4	"	Kingston, "	15.	3.39	1.60	1508.	217.21
5	"	" "	15.	3.85	1.57	1546.	203.64
6	"	Saugerties, "	14.	1.68	2.41	1702.	203.50
7	"	" "	15.	1.80	2.25	1385.	189.99
8	"	" "	13.	3.02	1.38	895.	168.59
9	"	Do. Mountain Gray	13.	3.05	1.55	1251.	184.95
10	"	Do. Brown Grit.	15.	3.40	1.82	1102.	122.31
11	"	Do. "	15.	2.18	2.00	*1000.	....
12	Marble.	Eastchester.	12.	2.20	1.27	535.	150.77
13	"	"	12.	2.19	1.27	508.	143.82
14	Sand Stone.	Belleville, N. J.	14.	1.48	2.97	990.	88.472
15	"	" "	14.	1.50	2.97	1000.	88.174
16	"	" "	14.	1.50	2.97	862.	76.006
17	"	Portland, Conn.	14.	1.48	2.98	1059.	94.004
18	"	" "	14.	1.38	2.97	761.	72.936
19	"	" "	14.	1.35	2.98	665.	64.715
20	"	Dorchester, N. S.	14.	1.45	3.10	7800.	66.980
21	"	" "	14.	1.43	3.15	781.	64.216
22	"	" "	14.	1.45	3.16	786.	63.333
23	"	Marietta, Ohio.	6.	2.03	0.97	240.	62.826
24	"	" "	6.	2.05	1.25	395.	61.659
25	"	" "	6.	1.43	1.58	401.	56.165
26	"	Berea, "	4.5	1.97	1.01	227.	42.359
27	"	" "	4.	1.63	1.03	200.	38.552
28	"	Amherst, "	14.	1.45	3.07	436.	37.221
29	"	" "	14.	1.47	3.05	410.	34.980
30	"	" "	3.	3.45	1.06	536.	34.568
31	"	" "	14.	1.47	3.05	395.	33.700
32	"	" "	5.	3.35	1.22	389.	32.506
33	Brick.	Colabergh.	7.	3.65	2.35	1435.	41.528
34	"	"	7.	3.65	2.35	1383.	40.023
35	"	"	7.	3.65	2.35	*900.	....
36	"	North River, hard.	7.	3.55	2.25	1317.	42.747
37	"	" "	7.	3.40	2.25	1129.	38.262
38	"	" "	7.	3.45	2.30	1080.	34.520
39	"	Phila., pressed.	7.	4.10	2.33	1597.	41.853
40	"	" "	7.	4.00	2.35	1223.	32.296
41	"	" "	7.	4.10	2.25	*800.	....
42	"	Staten Island.	7.	3.88	2.46	1692.	42.036
43	"	"	7.	3.88	2.37	1050.	28.104
44	"	North River, hard.	7.	3.60	2.35	1338.	39.259
45	"	" "	7.	3.60	2.35	1089.	31.953
46	"	" "	7.	3.60	2.35	830.	24.353
47	"	Perth Amboy.	7.	4.14	2.37	1051.	26.364
48	"	"	7.	4.12	2.33	1002.	26.132
49	"	"	7.	4.16	2.37	961.	23.991
50	"	"	7.	4.20	2.33	767.	19.662

* These broke at less than the weight stated.

TABLE II.

No.	Name.	Locality.	Specific Gravity.	Weight per cubic foot.	Percentage porous.	Percentage solid.	Action by Sulphate of Soda.	Action by Hydrochloric Acid.
1	Marble Dolomite.	East Chester, N. Y.	2.669	166.83	0.337	99.663	1	Dissolves.
2	Oolitic Limestone.	Aubigne, France.	2.331	145.67	7.758	92.242	3	"
3	Serpentine.	Chester, Penn.	2.301	143.80	8.452	91.548	1	Trifling.
4	Sand Stone.	Belleville, N. J.	2.269	141.83	8.528	91.472	4	"
5	" "	Middletown, Conn.	2.407	150.41	8.717	91.283	8	"
6	" "	Dorchester, N. S.	2.248	140.52	10.282	89.718	7	"
7	" "	Marietta, Ohio.	2.591	161.94	10.924	89.076	..	....
8	" "	Little Falls, N. J.	2.151	134.44	12.030	87.970	5	Trifling.
9	" "	Berea, Ohio.	2.146	134.14	13.851	86.149	6	"
10	" "	Amherst, Ohio.	2.131	133.16	14.561	85.439	12	"
11	Coquina.	Florida.	1.692	105.76	45.639	54.361	2	Slight.

TABLE III.

No.	Name.	Locality.	Weight per cubic foot.	Disintegration, (weight in grams per square inch superficial.)	Durability, (years required to disintegrate to the depth of one-tenth of an inch.)
1	Sand Stone.	Portland, Conn.	150.41	.0019706	2003.3
2	" "	Berea, Ohio.	134.14	.0017596	2000.8
3	" "	Marietta, Ohio.	161.94	.0023681	1794.8
4	" "	Dorchester, N. S.	140.52	.0045454	811.40
5	" "	Amherst, Ohio.	133.16	.0114040	306.46
6	Coquina.	Florida.	105.76	.4009300	6.9235

XLIX.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

EXPERIMENTS ON CEMENTS.

A paper presented by EDMUND YARDLEY, Civil Engineer, Member of the Society.

PREFATORY.

During several years past I have, from time to time, found it necessary to make experiments on the cements offered to me for use on the Pennsylvania Railroad; and during the past twelve months I have tried several brands in a more systematic way, the results of which, I trust, may not be uninteresting to members of the Society.

My primary object in communicating these results is to furnish data as to the strength of the cements experimented upon, that will be of use to members as means of comparison with other cements, they may have occasion to try; but incidentally I shall, I think, correct one or two errors into which, if I mistake not, many engineers have fallen.

The first and most serious of these errors, it seems to me, is to take simply, as to its setting or non-setting quality, the behavior of a ball of cement when immersed in water, for the true and only criterion of its value. Yet this is so frequently done, that I know of important works now going on, in which no other test of cements is made. I do not question that a cement, *not setting*, should be rejected; but it will

take but a casual examination of the Tables annexed to show great variations in value of those which pass this test, for I suppose all will agree that the true measure of the value of a cement is the *strength*, both cohesive and adhesive, it ultimately exhibits.

My experiments are defective in regard to this measure in two respects. First, I only have obtained the *cohesive* strength; and, second;—up to the longest time any mixture had been immersed, it was shown to be still increasing in strength. But they are not valueless on this account, for the cohesive strength is the more important element; and though the ultimate strength has not been obtained, the relative strength of the specimens cannot be far out of the way.

Another error of less moment, but still important, which is prevalent among engineers, is to assume that if two cements hold certain relations as to strength when pure, they will hold the same, or nearly the same relation, if each is mixed with an equal quantity of sand. A few experiments suffice to change this view.

My own idea as to obtaining the relative value of two cements is to mix one or both with sand until of the required strength, and then to calculate the values of the mixtures. Thus, a cement "A" costs 75 cents per bushel, and when mixed with two parts of sand, worth, say, 5 cents per bushel, it is equally strong with a cement "B," which sells for 40 cents per bushel. Evidently "A," costing, when mixed, one third of 85 cents, is the cheaper of the two.

I am aware that the experiments are incomplete, but I have tried to make what there is, accurate. Any result of which I had the least doubt has been rejected, unless it has served to indicate some point, and then its exact reliability has been noted. In some cases (especially the trials given in the first Table) subsequent experiments developed their incompleteness, though at the time they were considered reliable.

MANNER OF MAKING THE TESTS.

The manner of making the experiments was as follows: The cement under inspection, either pure or mixed with a proportion of sand, as the case might be, was made into a mortar as near[1] the consistency of

1. These were made *about* the consistency of ordinary mortar. Subsequent experiments seemed to indicate that very generally, though not universally, the less water that was used above a sufficiency to mix, the stronger the fusion. Hence, in the latter experiments, I have been as careful in measuring the water as the cement. —(See Experiments Nos. 24, 38, 97, et seq.)

that ordinarily used by workmen as possible, and was then worked with a spatula into a mould 1 inch square and about 8 inches long, placed horizontally[2]. When the prism had set sufficiently it was removed from the mould, and, after about 24 hours, immersed in water, (except in the case of some, in the earlier experiments, which were not immersed). At the end of about 30 days, the specimens were removed, and broken by a direct weight applied to the center between supports 6 inches apart. The broken halves being again immersed were, after varying intervals, broken between supports 3 inches apart.

REMARKS ON TABLE I.

The experiments in this Table (next page,) were carefully made, but there should have been more than one specimen of each tried, as faulty manipulation may have weakened a good cement.

I am sorry that I had only this test on the Louisville brand, which appears to be so weakened by the addition of sand. As this cement has borne a good reputation, I have asked the agents for another specimen, in order to satisfy myself of the above result being correct, though I have no reason—except there only being one prism tried—to doubt it.

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2. As the value of experiments increases with the number of others we are able to compare them with, it is proper for me to explain why I have proceeded differently from our standard American authority, General Gilmore, whom, it will be remembered, used a prism, 2 inches square, in a vertical mould, which set under a pressure of 32 lbs. per square inch, and was finally broken between supports 4 inches apart. My reasons briefly are these: at the time I made my first experiments, I did not have General Gilmore's book at hand; and when I afterwards procured it, for the purpose of making my experiments conform to his, I concluded to keep the size I had adopted, and to urge it for use to others, for the reason that to make the experiments with it requires no elaborate or expensive apparatus; all that I have used being an ordinary twine string to support the platform for the weights, and two pieces of boxwood scale for the supports. For breaking the 3 inch pieces I used an equally simple apparatus, which multiplied the weight applied eleven times. This of course, would be a very poor reason if any accuracy were sacrificed, but I cannot see that it has been. The 2 inch square prism contains more cement, but we have only to increase the *number* of our tests to get an equally reliable average. The horizontal mould I prefer to the vertical, as I believe it gives a fusion of more uniform strength. The pressure applied by him undoubtedly accords more nearly with practice; and should there be any reason to suspect that some cements were different relatively under pressure from what they are without it, I should consider it necessary to use it, even if it caused considerably more trouble.

Where more than one experiment is marked, the data have been partly derived from pieces broken between 3 inch supports, and reduced to what they would have been with 6 inch supports. An examination of the following Tables will show that there is no very close result obtained in this reduction, but I had to make it in this case, to condense and simplify the results from which the Table was obtained.

The Table shows the fallacy of judging of the strength of cements mixed with sand, from that of pure cements. In fact, in case of the Phœnix we see the unexpected result of an *improvement* in strength by the mixture, a result I felt doubtful about, until I found the same thing again in a cement, (the Allen) tried subsequently.

The results, as far as they go, also seem to indicate that it is an improvement to cements, to set under water rather than in the air.

SECOND SERIES OF EXPERIMENTS.

Guided by the experience derived from the earlier experiments, I used additional care in those which follow. In all cases I have made at least 3 prisms, of any particular mixture; and to guard against

TABLE I.

Breaking weights of one inch square prisms of different mixtures of cement and sand, between supports 6 inches apart. Prepared as noted above. Age 30 days. Results generally derived from a *single specimen*.

	Pure Cement.		Cement 2, Sand 1.		Cement 1, Sand 1.	
	Number of Experiments from which Average is obtained.	Average Breaking Weight. Ounces Avoirdupois.	Number of Experiments from which Average is obtained.	Average Breaking Weight. Ounces Avoirdupois.	Number of Experiments from which Average is obtained.	Average Breaking Weight. Ounces Avoirdupois.
Rosendale,[3] set in air	2	285	3	182	2	123
Louisville,[4] " "	2	220	2	61	2	37
" set under water	1	230	1	127	1	72
Phœnix,[5] set in air...	1	112	3	162	3	110
" set under water	..	....	2	185	..	....

3. "Rosendale," F. O. Norton, 91 Wall street, New York, from stock on hand; it was about a year old.

4. "Louisville," furnished by Laing & McKallip, agents, Pittsburgh; age unknown, probably fresh.

5. "Phœnix," purchased from J. R. Beesen, Uniontown, Fayette County, Pennsylvania.

TABLE II.

Breaking weights of one inch square prisms of different mixtures of cement and sand, between supports 6 inches apart,—loaded at the middle. Kept under water, after the time specified from mixing.

Kind of Cement.	Parts of Cement.	Parts of Sand.	No. of Specimen.	Proportion of Water to volume of Cement and Sand.	Hours in the air previous to immersion.	Age when broken. Days.	Breaking Weight. Ounces Avoirdupois.	Average Breaking Weight.	REMARKS.
ALLEN.[6]	1	0	22	Not measured.	About 24.	34	210.7	...	Top up.[7]
	"	"	23			"	250.1	241	Side up.
	"	"	24			"	263.8	..	Bottom up.[8]
	2	1	27	"	"	38	220.5	...	Top up.
	"	"	28	"	"	40	285.6	280	Side up.
	"	"	29	"	"	"	335.5	...	Bottom up.
	1	"	30	"	"	39	100.9	...	Top up.
	"	"	31	"	"	"	162.4	162	Side up.
	"	"	32	"	"	"	223.3	...	Bottom up.
	"	2	33	"	"	38	103.0	...	....
	"	"	34	"	"	"	75.3	101	....
	"	"	35	"	"	"	124.9	...	....
ROSENDALE.[9]	"	0	36	"	"	37	249.0	...	....
	"	"	37	"	"	"	250.9	240	....
	"	"	38	"	"	"	220.0	...	Mortar very thin.
	2	1	57	"	"	28	199.7	...	....
	"	"	58	"	"	"	160.6	181	....
	"	"	59	"	"	"	181.9	...	....
	1	"	72	About 33 per cent.	"	32	140.5	...	....
	"	"	73		"	"	147.3	146	[10] ....
	"	"	74		"	"	150.5	...	....
	"	2	64	Not measured.	"	"	48.6	...	....
	"	"	65		"	"	45.6	50	....
	"	"	66		"	"	55.3	...	....
DIAMOND.[11]	"	0	39	"	"	35	393.3	...	....
	"	"	40	"	"	"	561.0	468	[12] ....
	"	"	41	"	"	38	451.3	...	Thinnest of the 3.
	2	1	52	"	"	40	164.9	..	" "
	"	"	53	"	"	"	304.6	271	....
	"	"	54	"	"	"	343.1	...	....
	1	"	69	"	44.	31	194.6	...	....
	"	"	70	"	"	"	214.1	213	....
	"	"	71	"	"	"	229.2	...	....
	"	2	61	"	About 24.	32	39.4	...	....
	"	"	62	"		"	59.3	61	....
	"	"	63	"		"	83.9	...	....
ROSENDALE.[13]	"	0	97	.44	20.	33	168.5	...	....
	"	"	98	.37	"	"	194.2	194	....
	"	"	99	.37	"	"	220.4	..	....
	2	1	111	.28	48.	"	243.6	...	....
	"	"	112	.31	"	"	247.9	240	....
	"	"	113	.31	"	"	230.4	...	....

6. The "Allen" cement is made at Easton, Pennsylvania. Berger & Butz, agents.

7. Care was taken to notice which side of the specimen as made in the

error from the use of different kinds of sand, I employed a sand made by crushing the stone. This is easily obtained here, and is entirely free from loam. It is properly comparable, I should say, with the white sand used so extensively as a scouring material. It was passed through a sieve, of about 18 meshes to the inch, to separate the coarser pebbles, which might, if located at the point of breaking of the prism, vitiate the result. The cement was not sifted, as my desire was to get at its strength as it was, not as it might be if ground finer.

Additional care was taken to have the cement and sand thoroughly mixed when dry, which was done, from experiments Nos. 30 to 48, by repeated (3) siftings. To save trouble, from the last to No. 72, they were merely thoroughly shaken together in a box; but as the strength of the specimens departed more widely from the average, I returned at No. 72 to the former plan, which I still use.

In other respects the manner of proceeding was the same as that already described for the first experiments.

REMARKS ON TABLE III.

For this Table, the halves of the prisms already tested (per Table II.) were broken again, between supports 3 inches apart. Therein is shown the gradual increase in strength of the cements up to 6 months for many of the cements, and we are enabled to judge how far trials made when the prisms are 30 days old will give us the actual relative value of the cements when they have attained their greatest strength.

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mould was up, (that is, exposed to a compressive strain when broken,) but after a few experiments, all were tried on the side as that seemed to give an average result. All after No. 33 were tried in that way.

8. Mortar was stiffer for this than for Nos. 22 or 23.

9. The first sample of this cement—that without sand—was received by the Pennsylvania Railroad Company in June, and the trials were made in October. That used in the other three mixtures was from a different barrel, about the same age.

10. In all of the experiments a larger or smaller quantity of a light flocculent matter, I supposed to be carbonate of lime, is found in the water. In this case there was a great deal, though the "Rosendale" generally had as little as any. In one case, which I have omitted for want of room, the water was very full, and the prism was very light and weak; I think, indeed, most of the lime was dissolved out.

11. "Hydraulic Diamond Cement." J. V. Devling & Co., Flemington, Clinton County, Pennsylvania; from stock purchased.

12. These prisms were bowed up somewhat when taken from the water, and seemed cracked.

13. From a new barrel; the cement about 10 months old.

TABLE III.

Breaking weights of the prisms (Table II.), between supports 3 inches apart, for different ages up to 6 months. Prisms kept under water the whole time.

Kind of Cement.	Parts of Cement.	Parts of Sand.	No. of Spec.	30 days.		60 days.		90 days.		6 months.		
				Exact age.	B. Wt. Ounces.	Exact age.	B. Wt. Ounces.	Exact age.	B. Wt. Ounces.	Exact age.	B. Wt. Ounces.	Average at 6 months.
ALLEN.	1	0	22	..	....	68	984	...	....	188	1458	
	"	"	23	35	576	..	....	106	969	...	....	1458
	"	"	24	35	627	..	....	...	....	...	....	
	2	1	27	40	658	67	941	..	...	187	1625	
	"	"	28	..	....	..	....	105	1020	"	1349	1424
	"	"	29	..	...	..	....	...	....	"	1299	
	1	"	30	..	....	67	571	...	....	186	905	
	"	"	31	..	....	..	....	104	620	"	859	286
	"	"	32	..	....	..	....	...	....	"	715	
	"	2	33	..	....	62	366	...	....	182	607	
	"	"	34	..	....	..	....	99	505	"	525	637[14]
	"	"	35	..	....	..	....	...	....	"	703	
ROSENDALE.	"	0	36	..	....	61	755	...	....	"	1093	
	"	"	37	..	....	..	....	97	1013	"	1200	1073
	"	"	38	37	440	..	....	...	....	"	925	
	2	1	57	40	558	..	....	...	....	"	1761	
	"	"	58	..	..	..	....	95	992	"	1631	1756
	"	"	59	..	....	58	725	...	....	"	1876	
	1	"	72	..	....	..	....	...	....	...	....	....
	"	"	73	..	....	..	....	...	....	...	....	....
	"	"	74	33	297	..	....	...	....	...	....	....
	"	2	64	32	159	..	....	...	....	174	698	
	"	"	65	..	....	..	....	96	319	"	644	650
	"	"	66	..	....	63	215	...	....	"	607	
DIAMOND.	"	0	39	..	....	..	....	93	1520	190	1356	
	"	"	40	..	....	..	....	...	....	"	1156	1377
	"	"	41	42	1414	64	1376	...	....	189	1620	
	2	1	52	43	426	..	....	...	....	...	1102	
	"	"	53	..	....	..	....	90	1120	185	1550	1457
	"	"	54	..	....	60	874	...	....	...	1719	
	1	"	69	..	....	..	....	98	1069	...	....	....
	"	"	70	..	....	61	776	...	....	...	....	....
	"	"	71	33	522	..	....	...	....	...	....	....
	"	2	61	32	86	..	....	..	....	175	342	
	"	"	32	32	130	..	....	97	369	"	....	523[15]
	"	"	63	..	....	64	357	...	....	"	704	

14. Specimen No. 34 was cracked, and its value as an experiment was taken at one half the others. The other half of No. 35 bore 656 ounces, hence the average as given.

15. Specimen No. 61, at 6 months' age, broke with the first weight, 342 ounces. The average is not *over* the amount (523 ounces) stated.

CONCLUSION.

From these Tables we see:—

First. That a mixture of as much as one third sand does not diminish the ultimate cohesive strength of any of these cements, but in some cases increases it.

Second. Comparing the three cements together at the end of 6 months, there seems but little difference in their values; the strongest product being "Rosendale" mixed with one third sand.

Third. While at 30 days we can form a close approximation as to the ultimate relative strengths of cements, it requires a much longer time (probably over 6 months) to determine it exactly.

I.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

CLOSING BREAKS IN CANALS, UNDER DIFFICULTIES.

A paper read by O. T. WHITFORD, Civil Engineer, Member of the Society.

The Binghamton dam and guard lock are situated on the north branch of the Susquehanna river, where its course is nearly west. The dam has a spillway of 350 feet, and a fall of 6 feet. At its north end are a bulkhead and feedgates to several mills and factories, and at its south end, the feedgates of the extension of the Chenango Canal and a guard lock, situated near the river bank and parallel to it, with the lower hollow quoin opposite the crest of the dam.

The lock stands upon a rock bottom, and its walls are 18 feet high. At the time of high water, in 1872, this structure was barely finished. The rise of the river was about 13 feet; the highest in seven years, or since the spring of 1865.

During the night of April 8th, 1872, the coffer dam around the lock gave way; the embankment behind the lock, which had been hastily put in during the previous frosty weather, soon followed, and the river opened a passageway around the lock. As the damage was done during a dark and rainy night, neither men nor materials could be procured to close the break.

The river bank, where the shore line was cut away, proved to be a loam formation, resting on quicksand, and was rapidly removed, so that

when we were prepared to close the break there was an opening 60 feet wide at the head of the lock, with the water varying from 6 feet deep at the shore to 10 feet deep at the lock, and a fall in the length of the lock equal to that of the dam.

The means taken to close the break were simple, yet effectual. Three snubbing posts were set along the river bank, the first at the head of the break, the others up stream about 30 feet apart. A bundle of two or three wagon loads of brush was made up, by alternately reversing the tops so that it could be firmly bound around the middle; it was then rolled into the water at the point where we proposed to close the break, and tied to the snubbing posts; a two-inch Manilla rope was sufficient for that purpose. The bundle was then loaded with stone until it rested firmly upon the bottom, and alternate layers of brush and stone built up above the surface of the water; care was taken to make a good connection with the shore by channeling into it and tying across with fine brush and gravel. A second bundle was anchored outside the first and loaded in the same manner, and the process continued until we reached the deep water. When it became necessary to strengthen the wall, this was done by building to the down stream side. We also launched our bundles from the upper corner, letting each one project a little beyond the preceding one; we pushed them out "en echelon," so they could rest against the wall until they sank to place; thus we were enabled to hold them in position, which we could not have done if they had swung out into the current. As our wall approached the lock, it was turned down and built a short distance parallel to the lock walls, then a large bunch of brush was wedged in and loaded until it was fairly driven home. The brush and stone were afterwards built up to high water mark. A few loads of gravel, dumped over the face, tightened it up, and an embankment, 25 feet wide, was built behind and carried up 5 feet above it and faced by a slope wall on top of the brush and stone.

The closing of the break by this plan proved simple, cheap and permanent. The brush was obtained from a neighboring fallow, the stone picked up from the fields. Thirty men were about as many as could be worked to advantage, with a dozen teams bringing material.

The material for the embankment was loam and gravel intermixed, and impervious to water; it was hauled on in wagons. The brush and stone served as a permanent protection against further encroachment by the river.

LI.

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AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

THE MANUFACTURE OF COKE FROM ILLINOIS COAL.

A paper presented by HENRY L. LUEBBERS, Civil Engineer, Member of the Society.

The coal of Western Illinois, owing to the great amount of impurities occurring with it, may, with a few exceptions, be considered unfit for the iron smelting furnace, as well as for all other purposes requiring a fuel free from sulphur, and leaving but a small percentage of ash when burned. This coal is bituminous and mined without difficulty; the nearly horizontal beds are easily reached by shafts; the layers are stratified, and seams of dull and glossy appearance alternate. The maximum thickness of the beds is about 7 feet. Bituminous shale, fire-clay, carbonate of lime (or gypsum ?), and pyrites of iron appear with the coal in layers or seams; occasionally a thin seam of mineral charcoal is found. The pyrites of iron, where they occur in layers parallel with those of the coal, vary from thin scales to seams of 2 inches or more in thickness; their seams frequently run vertically or diagonally across the layers of coal; carbonate of lime appears only in thin seams, and running parallel as well as at right angles to the layers of coal. Fire-clay and bituminous shale form the bottom, and the latter sometimes the roof of the coal beds.

The coal itself is rather brittle, and on an average there results about 10 per cent. of slack in mining. Of late years this slack coal is made use of in the manufacture of coke, after subjecting it to a washing

process, and the experiments made on a large scale, although still in their infancy, are quite promising.

In the following I shall give a brief description of this process. I must remark here that the slack coal is preferred for coke making on the ground of economy; the pyrites of iron contained in the bed coal cannot be removed by even the most careful picking, and consequently it could not be made to yield a good coke without being purified; hence the expense of preparing the bed coal for the ovens would be about the same as in the case of slack coal.

According to the character of the mine and the care bestowed on mining and picking the coal, the percentage of pyrites and other impurities in the slack coal differs widely; from sixteen to twenty per cent. of ash may be considered a fair average; whole well picked and selected specimens of lump coal show from two to six per cent. of ash.

The slack coal, before subjecting it to the washing process, is crushed in a mill or between grooved rollers to obtain the desired fineness, which will depend entirely on the character of the coal. The crushing should be done to such an extent that the particles of the pyrites of iron (sulphur, in mining parlance), carbonate of lime, and slate are separated from the coal as much as practicable. If the crushing is carried too far, the cost of the washing process will be increased materially, and a large loss from dust will result.

The crushed coal is then classified according to size in a revolving drum, covered with wire netting or perforated iron plates, and divided into several compartments. Each compartment has a different class of holes; the dust is generally screened off first, and the coarsest coal last. Sometimes a special drum is employed to screen off the dust which, requiring an expensive arrangement of dust washing jigs and large settling reservoirs as well as a great deal of manual labor, is frequently allowed to pass off to the waste pile with the impurities. The screening may also be performed in a reverse order by sifting off the coarsest coal first and the finest coal last. This arrangement requires more height and a system of several drums, but it allows a more perfect classification of the smaller sizes.

It is essential for a thorough washing process to divide the coal into as many classes as possible; in practice probably six sizes will suffice for the most impure Illinois coal. The percentage or amount that may be expected of each class or size, and the requisite width of the holes in the wire netting, can only be determined by a previous experiment in each particular case.

The classified coal goes to the washing machines, which are built after the pattern of the mining jig, although of larger sizes than are usually employed for the washing of ores. The materials to be separated differ sufficiently in specific gravity (pyrites of iron from 4.5 to 5; bituminous shale and fire-clay 1.9 to 2.2; coal 1.2 to 1.3), to allow a perfect separation; however, as mentioned above, pyrites, slate and coal are frequently so interlaminated that the product has to be washed over twice; this is done by coupling together two jigs and running the washed material from the first jig through the second one, to take out the impurities that might be left with the coal from the first jig. The stroke of the jigs and the number of strokes depend upon the size of the material, the coarser sizes requiring a larger stroke and a less number of strokes than the finer ones. The stroke varies from 1 to $2\frac{1}{2}$ inches, and the number of strokes from 70 to 110 per minute.

A thorough separation requires water pure in quality and ample in quantity. I will remark here, by the way, that to wash eight tons of slack coal per hour requires at least 12,000 gallons of water.

The purified coal is collected from the different jigs in a gutter and led to a draining drum, of a construction similar to that of the classifying drum, and revolving at about the same speed (10 to 12 revolutions per minute). This drum is covered with a wire netting, fine enough to retain the coal, which is then crushed again between smooth rollers to obtain a more uniform material and to produce a coke of greater density.

The washed coal is coked in retort ovens, which were introduced several years ago in Pennsylvania, and are now about to supersede the so-called Pittsburg ovens. Their advantage is to require but little manual labor. They are of the shape of an arched culvert, from 18 inches to 6 feet in width, according to the character of the coal, closed at either end by cast iron doors with fire clay lining. They are built in rows of twenty to thirty ovens. The process of coking lasts from twenty-four to thirty-six hours. The doors are then raised by means of cranes, or swung around on a pivot, and the coke pushed out by a ram, worked by a movable steam engine, traveling on a track parallel to the ovens. In some instances the coke is drawn out by means of a cradle and chain worked by a stationary engine; it is evident, however, that this plan is inferior to the former, considering that the cradle and chain will have to remain in the oven during the coking process.

These ovens do not require any artificial heat except at the beginning of a run, the burning gas passes through flues in the side-walls and under the bottom of the ovens, thereby heating up the body of fire-brick sufficiently to kindle and coke the next following charge, which, in its turn, again prepares the oven for the subsequent charge. It would lead toò far to go into a minute description of these ovens and the different patented and non-patented arrangements of the flues. The main requisites with them are a sufficient but not too strong draught, so as to utilize all the heat generated in the combustion of the gases, and a body of fire-brick sufficiently large to receive and sustain this heat until the following charge is put in.

The coke product is somewhat porous but sufficiently dense for the blast furnace; it weighs about 38 pounds per bushel. The loss in washing varies from 20 to 30 per cent., according to the amount of impurities contained in the slack coal and the hardness or brittleness of the coal. The yield of coke is about 60 per cent. of the weight of the washed coal.

LII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

AT THE FOURTH ANNUAL CONVENTION OF THE SOCIETY, HELD AT CHICAGO, JUNE 5TH AND 6TH, 1872.

SELECTION OF STONE FOR MASONRY.

A paper presented by GEORGE R. EICHBAUM, Civil Engineer, Member of the Society.

The subject of the present remarks is so intimately connected with the permanence of structures, that it is unnecessary to enlarge upon it, nor is it necessary to allude further to the durability of pyramids and other works of the ancients, which (in many cases monuments of vain folly), are all objects alike of wonder and admiration.

Coming down to times nearer, in the object and style of their works, to the present, we find that the men who planned and executed the great works of Europe, in their exertions to procure suitable material, were groping in darkness during the early stages of geological science and the fierce discussions of the Huttonian and Wernerian theories. For instance, in England, the works of Messrs. Brindley and Smeaton were constructed before Mr. Smith presented his theory and system of the stratification of England. But this article is intended to bear upon the subject in regard to the works of the United States, and to them it shall be limited.

The great works, among which were the Union, Chesapeake and Delaware, Chesapeake and Ohio, Erie, Pennsylvania, Ohio, the Indiana, Illinois, and Louisville canals, were commenced without aid from the geologist, excepting that derived from the general observations of Featherstonhaugh, Houghton, and Schoolcraft. The Pennsylvania

railroad had some benefit from the surveys of Rogers, but the earlier works were attended with much uncertainty and difficulty in procuring suitable building stone, because of the want of systematic examination of the stratification. This was apparent also in the buildings erected in cities, quite as much as upon locks and other masonry of public works. Yet, while nearly every line of work has shown the effects of carelessness, cupidity, or want of comprehension of the subject, many works have left testimony of the exertions and success of their builders in securing sound material. The great National road, useful in its day, presented many lasting structures, so have the Baltimore and Ohio, the Pennsylvania and other railroads; and Messrs. Roebling, Ellet, Fink, Eads and others, while making sure of foundations and arranging superstructures, needing less bulky piers and abutments, realized the importance of sound material for the intervening masonry.

Defective material seems to be the cause of the ruin of buildings in nearly all of our cities; in New York, occasioned partly by the dampness and the muriatic acid with which the atmosphere is charged, and in Pittsburgh by the peculiar situation of that city, enshrouded in smoke and exposed to the action of sulphurous vapors.

Now the question naturally arises, what can be done to remedy this evil, or prevent the continuance of it? The only sure test of the durability of stone, in searching for material, has been considered by many to be the examination of old structures, the stone of which was from a particular stratum, or of the face of rock where the natural ledge is exposed, but neither of these is infallible in every case, as the texture of the material of the stratum varies sometimes within short distances, and stones very similar in appearance sometimes vary in durability. In fact, as one of our most eminent authors remarks, stones having the same texture and chemical composition are, from causes not fully ascertained, of very different degrees of duration; while the sudden and extreme changes to which the climate of this country is liable can only be withstood by stone of proper components and texture, and these are to be tried by various tests.

Many of the States have had, within a few years past, geological surveys made for the benefit of various interests. Those surveys have proved, in many instances, beneficially available to the coal and iron interests; and engineers and others needing suitable building stone, may find it not only desirable but expedient to examine the data furnished by geological surveys, and by connecting their own notes

of level and position of rock with those of the geologist, can save much time and useless examination in search of material. Often the description and chemical analysis of the geologist, if accompanied with evidence of old structures or appearance of natural ledge, may be decisive as to the character of the rock; otherwise, a fresh analysis should be made and the usual tests of the absorbent quality of the stone, its liability to effects of frost, etc., also be tried.

The geologist usually gives a clear description, whether the stone be any variety of granite, limestone, sandstone, etc., but he does not designate the application to which particular kinds of rock are suited; while some stones, especially of the porous limestones and sandstones, are good when kept dry, but unfit if exposed to rains and prevalent winds, or to being alternately wet and dry, where subject to rise and fall of water.

I trust that enough has been stated to draw attention to a matter so vital in its consequences.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

AT A MEETING OF THE SOCIETY, JUNE 16TH, 1869.

ON THE COMPOSITION OF ANCIENT CEMENTS AND ROSENDALE CEMENTS.

A paper read by ARTHUR BECKWITH, C. E., Member of the Society.

The cements commonly used in New York and elsewhere for ordinary constructions as well as for large engineering works, have frequently elicited the interest of inquirers into their nature and properties.

Among the best descriptions of the qualities of these cements we may cite General Gillmore's work on the subject, in which we find a list of valuable experiments which set forth the properties of various cements, and also a table showing the chemical composition of a number of cement-stones from which our cements are made.

These experiments and tables do not, however, contain the chemical analysis of the cements themselves, upon which so much depends.

The chemical analysis of several of the Rosendale cements has lately engaged my attention, but before setting forth the results obtained I will recall briefly the history of our modern knowledge of cement.

The monuments of Egypt present one of the oldest examples of the use of lime for constructions. The mortar which joins the stone of the Pyra-

mid of Cheops is precisely similar to modern mortars made of sand and lime. In limiting the use of mortar to filling narrow joints which separate immense blocks, and thereby reducing almost to insignificance the part which it had to play, the Egyptians seemed to forestall the influence of a dry and burning climate. Time has justified their prudence in this respect, for the works erected on the banks of the Nile by the Romans, made of small materials and presenting many joints, have left but faint traces, whilst some Egyptian temples still present themselves intact to our admiration.

Unqualified praise has often been given to the excellence of Roman mortar, and the belief is sometimes expressed that all we can hope to do is to regain the secret of making mortar once possessed by the Romans. It is a common remark that "Roman mortar has lasted for eighteen centuries, whilst a number of modern buildings are in a deplorable state of preservation."

To make a fair comparison, we should, however, only cite similar constructions, and then we are comforted by these words of Pliny: "The cause which makes so many houses fall in Rome, resides in the bad quality of the cement."

The knowledge of the properties of lime descended from Egypt to Greece, where the exigencies of the climate and the ingenuity of the people brought forth many of its uses, unknown to Egypt.

Subsequently Greek colonies imported and popularized their processes in Italy; and Roman architects, like Vitruvius, cite the names of Greek authors on the art of construction. Their names alone have come down to us, but Vitruvius had full access to them, and in our inquiry after the knowledge of mortar possessed by the Romans, it is to him that we must refer for information. Indeed he has left us a detailed table of precepts used by the builders of Greece and Rome, which do not justify our unreserved admiration; everything relating to lime, sand, and pozzolana is clearly treated therein.

We may safely affirm, with Vitruvius, that the Romans made use of the lime, sand, and materials of the countries where they built; that they considered the best lime to be produced from hard and pure marble, *i. e.*, the fattest lime known; that in Italy they mixed it with pozzolana when used for hydraulic purposes, and that out of Italy they replaced the pozzolana from Vesuvius, by powdered brick or tile.

Roman mortars, when examined to-day, are found to bear a distinct

resemblance to each other; they may be recognized by the presence of coarse sand mixed with gravel; lumps of lime are so often to be met with, that incomplete slaking will alone account for them. Mortars laid in damp spots for cisterns and pavements were composed of bricks in small fragments mixed with fat lime; this concrete required to be compacted by pounding and left to dry—the surface was then scraped, polished, and painted—evidently to prevent the dissolution of lime by water.

It will be seen by this that what we term hydraulic lime, and also the modern product of cement, were unknown to the Romans.

It is important to refute the belief that methods may have been known to them of which we have lost the secret. When the decay of arts followed upon the downfall of the Roman Empire, houses nevertheless continued to be built, and the familiar processes under the eye of the workman must have been transmitted from father to son. So true is this, that to-day Italian masons, who certainly have not read Vitruvius, make coatings for cisterns and concrete floors in the very same manner as may still be seen in the ancient ruins of Rome

Neither is it true that Roman mortar is uniformly good. Its strength of cohesion varies in different examples from 35 and 85 lbs. per square inch to 100 and 160 lbs., or as much as 500 per cent.

In the middle ages a volcanic conglomerate from the banks of the Rhine, named traass, was substituted for the pozzolana of Italy, and mortar was made of fat lime, mixed with traass, to render it hydraulic.

Many castles erected during that period stand well to-day; the well-known castle of the Bastile, erected in 1369–83, which after withstanding a siege required the use of powder for its destruction in 1789, was found to be extremely solid even in the interior walls.

It would seem, then, that the secret of the Romans was known also in those times, and could have been lost only at the Renaissance, when least of all such a supposition is probable.

At what period were first used certain limestones, having the property of producing a lime which will harden under water, is not precisely known; the first use of cement stone is equally obscure.

In 1796 Messrs. Parker and Wyatts began to manufacture from egg-shaped limestones found near London, a product known later as *Roman Cement*, and which was soon received with great favor throughout Europe; but neither the producers nor the consumers offered any explanation of its merits.

Not until 1818 and the following years was the true explanation given of the hydraulic properties of limes and cements, when Vicat published his discoveries.

Before that, in 1756, when Smeaton was preparing the arduous and bold construction of the Eddystone Lighthouse, this celebrated engineer examined with scrupulous attention the natural hydraulic lime of Aberthaw. Treated by acids it left a residue "which appeared to be a bluish clay, weighing about one-eighth of the total weight of the stone."

In 1786, Saussure attributed the hydraulic properties of some limes of Savoy to the combined influence of manganese, quartz, and even clay; but he left his opinions in the mere state of conjectures.

Finally, Descostils, in 1813, having discovered a considerable proportion of finely divided silica in the lime of Senonches, attributed the well-known hydraulicity of that lime to the silica it contained.

But the conjectures of Smeaton, of Saussure and of Descostils were vague; they rested upon no proofs, and found no applications in practice.

The discoveries of Vicat attained their immediate object, for in a short time artificial hydraulic lime of excellent quality was manufactured on a large scale under his direction, and a few years later he indicated as many as 400 quarries in France where hydraulic limestones were to be found.

Moreover, the mortar made from his hydraulic lime equalled in hardness at the end of eighteen months the hardest ancient Roman mortars.

It is unnecessary to recall the evidence by which Vicat demonstrated —by analysis and by synthesis—his great discoveries. No one questions to-day the fundamental truth, that the properties of hydraulic limes depend upon the proportion of clay disseminated throughout its tissue, and that clay by being calcined acquires the property, like pozzolana or traass, of rendering fat limes hydraulic, when thoroughly diffused throughout their mass.

The labors of Vicat and Berthier have led to the following classification of limes and cements, and consequently of limestones and cement-stones.

Table of Classification of Limes and Cements.

Proportion of clay in the limestone.	Proportion of clay in the product.	Class of lime or cement.
Less than 10 per cent.	Or less than 17 per cent..	Fat and non-hydraulic limes.
From 10 to 15 " ...	From 17 to 24 " ..	Slightly hydraulic limes.
" 15 to 17 "	" 24 to 27 " ..	Hydraulic limes.
" 17 to 20 " ...	" 27 to 30 " ..	Eminently hydraulic limes.
" 20 to 23 " ...	" 30 to 34 " ..	Limit of hydraulic limes.
" 23 to 30 " .	" 34 to 43 "	Beginning of cements.
" 36 per cent....	" 50 per cent........	Good hydraulic cements.
" 40 per cent	" 54 per cent.......	Hydraulic cements of diminishing value.
" 60 to 90 per cent..	" 73 to 94 per cent...	Pozzolanas.

A point which bears directly upon our subject is the fact of the existence of a limit for the proportion of clay, at either end of the scale of cements. The transition from the properties of hydraulic lime to those of cement is not gradual, but sudden. Thus a limestone containing 20 per cent. of clay will produce an eminently hydraulic lime, but if we increase this proportion to 23 per cent., it is neither a hydraulic lime nor a tolerable cement that we have, but a worthless product, which if submerged will remain for days and even weeks without giving any sign of slaking, and then crumble away insensibly without effervescence; or if pulverized and tempered like plaster, will give an appearance of setting, but crack and turn into mud when submerged.

These products, which may be called the intermediate limes, are found on an average between 20 and 23 of clay for 100 of limestone; but these numbers are not absolute, for some limestones containing 21 and 23 per cent. of clay make both good hydraulic lime and cement, and also the former when underburnt give very irregular results, forming sometimes a cement and at others a worthless compound.

In the same way there is a superior limit to the proportion of clay in cement, which when surpassed gives a poor cement. The exact position of this superior limit is not entirely agreed upon. It is placed at 36 per cent. and sometimes 40 per cent. by Vicat, and at 40 or 46 per cent. by Berthier.

The composition of the layers forming the quarries from which the Rosendale cements are taken is extremely variable, the proportion of clay ranging from 15 per cent. to 47 per cent. Some of these layers contain the right proportion of clay for good hydraulic limes, and for cements, while others contain the proportions which correspond to the intermediate limes and the superior limit of cements. The separate layers are not entirely uniform in their composition, and, like all beds of limestone, those situated near the surface lose a portion of their carbonic acid by the alternate action of heat and moisture.

Therefore, if the stones obtained from the different layers be mixed according to color and physical appearance, as is sometimes practised, and without a due regard to the exact chemical composition of each, it is obvious that uniform and good results are not likely to be obtained.

I am unable to give at present the result of the analysis of more than four of the different brands of Rosendale cement which I have examined, and the labor being incomplete, I refrain for the present from naming the brands which have been analyzed.

The following are the results:

Analysis of Cement No. 1.

Water	1.2 per cent.		
Carbonic acid	traces.		
Silica	30.7	per cent.,	or as 43.6 per cent. of clay to 56.4 per cent of lime and magnesia.
Alumina and sesquioxide of iron	12.0		
Lime	42.6	per cent.,	
Magnesia	12.8		
Total	99.3		

Analysis of Cement No. 2.

Water	0.2 per cent.		
Carbonic acid	traces.		
Silica	33.	per cent.,	or as 46 per cent. of clay to 54 per cent. of lime and magnesia.
Alumina and sesquioxide of iron	13.		
Lime	33.	per cent.,	
Magnesia	20.		
Total	99.2		

Analysis of Cement No. 3.

Water......	0.5 per cent.		
Carbonic acid	traces.		
Silica	27.	per cent.,	or as 37 per cent. of clay to 63 per cent. of lime and magnesia.
Alumina and sesquioxide of iron.	10.		
Lime	50.3	per cent.,	
Magnesia	12.		
Total................	99.8		

Analysis of Cement No. 4.

Water......................	0.2 per cent.		
Carbonic acid	traces.		
Silica.................	31.6	per cent.,	or as 40 per cent. of clay to 60 per cent. of lime and magnesia.
Alumina and sesquioxide of iron	7.8		
Lime and magnesia	60.	per cent.	
Total....	99.6		

These results show that the Rosendale cements above examined, contain a proportion of clay which approaches, in some cases, to the proportion indicated by Vicat as forming the best cement, and in others to a proportion nearer the beginning of the scale of cements.

A point worthy of notice is, that if we compare these cements to the English and French cements, the one marked No. 3 contains nearly the same proportion of clay as the French Portland; No. 4 contains the same as the cement of Vassy; Nos. 1 and 2 contain more clay, although nearer the proportions named by Vicat for the best cements, and all contain more magnesia than is common to European cements.

The cements examined also contain traces of alkalies and chlorides. One contained $\frac{7}{1000}$ of sulphate of lime, which is not to be considered injurious, as it does not exceed three per cent.

The large proportion of magnesia in these cements is remarkable. Chemists are not wholly agreed upon the effects of magnesia in the presence of lime.

Magnesia in the presence of silica and alumina is known to form crystallizations which resist the action of sea water better than lime—and

Vicat remarks that the presence of magnesia exalts the quality of cement for marine uses.

On the other hand, it is equally certain that the silicate of magnesia crystallizes slower than the silicate of lime, and Rigot asserts that the consequence of the presence of magnesia is disaggregation or at least inferior hardness.

In the presence of these conflicting opinions the true influence of magnesia remains a subject for investigation.

Having but recently analyzed various American limes and cements, I am not able to present comprehensive or complete results, and my object in introducing the subject at this stage is to call the attention and invite the labor of others, in completing the studies required for the uniform production of the best quality of hydraulic limes and cements. But my inquiries have gone far enough to convince me that standard cements will not result from experimental mixtures, not guided by selections based upon accurate analysis.

I conclude with the following analysis of Rockland lump lime:

Water and carbonic acid........	traces.	
Silica........................	5.6	per cent., or 7.8 per cent. clay
Alumina and sesquioxide of irom	2.2	
Lime.........................	87.6	per cent., or 91.9 per cent. lime and magnesia
Magnesia....................	4.3	
Total................	99.7	

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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AT A MEETING OF THE SOCIETY, MARCH 20TH, 1872.

THE MEXICAN METHOD OF MAKING HARD LIME FLOORS.

A paper presented by Gen. THEO. G. ELLIS, C. E., Member of the Society.

Some years since, the writer had occasion to visit Northern Mexico to examine and work some silver mines; and, while passing through the Mexican villages, noticed the exceedingly hard and polished lime floors and roofs of the houses.

In the village of Salinas, where our party remained some three weeks, the horses passed daily through the house into the inner court-yard over one of these floors without leaving an indentation, or injuring it in any way. Some time after, having occasion to construct some buildings at La Yguana mines, an attempt was made to imitate these floors and roofs. Attributing the peculiar hardness and smoothness of the floors to the inherent good qualities of the lime used, no inquiries were made as to the Mexican method of working. A good quality of limestone was selected and calcined in the ordinary way. Shortly after burning it was slaked to a dry powder, and afterwards used as required. A floor was laid with a foundation of about three inches of broken stone, over which was evenly

spread about two inches of mortar, formed of two parts of clean sharp sand, and one of lime. The lime was "fat," swelled greatly in slaking, and was not at all hydraulic.

The floor, made as above, was a total failure. At the end of four weeks the leg of a chair would indent it. As soon as the surface was damaged it began to crumble; and soon broke up. It would probably have been about as hard as our ordinary lime mortar if allowed to set a sufficient length of time before being used.

Knowing that the Mexicans used the same materials with better success, their superior skill was called into requisition to lay all the remaining floors and roof of the same building. They used the same sort of lime and sand in about the same proportions, and upon the same kind of foundation. The result was a floor as hard and smooth as a piece of polished marble, that a horse could trot upon without injury.

A brief account of the method of making these floors may not be uninteresting.

The limestone used was a hard, compact blue material, in some places sufficiently hard to strike fire on the drills used in running a drift through it for mining purposes. It often contains iron pyrites in small proportion. This was calcined in kilns cut out of a very soft limestone, that likewise is found in that section of country, and which on account of its whiteness and softness is called "cal leche." I believe it is never used for making lime by the Mexicans.

After calcination the lime was removed from the kilns and slaked as soon as cool. Some of it was used within a day or two, and some remained a month or more in barrels. All the work made with it seemed to be equally good.

In making the floors, a layer of broken limestone, three or four inches thick, was first laid evenly over the surface of the ground. The stone being about the usual size for macadamizing roads; over this a mortar of about two parts of sand to one of lime was carefully and evenly spread to the thickness of one and a half to two inches; this was allowed to remain for about twenty-four hours, or until the surface had become quite dry. It would probably take longer in this climate, where the air possesses a greater amount of moisture than in Mexico.

The floor was then thoroughly pounded all over with a tool composed of a block of wood about a foot square and three inches thick, having a

handle rising from the middle, so that a man could stand while using it. The whole surface was beaten over with this ram until it was again as soft and moist as when first laid. This operation of ramming brought the water in the mortar to the surface so as to form a layer of semi-liquid substance on top.

The floor was again allowed to dry, and again beaten over each day for about a week, when the operation brought only a slight amount of moisture to the surface.

Immediately after the last pounding the whole surface was powdered with a thin layer of red ochre, evenly sifted on, and then polished as follows:

A smooth, nearly flat, water-worn stone, a little larger than the fist, was selected from the bed of the stream which ran through the place, and with this the whole floor was laboriously gone over, rubbing down and leaving the surface of the lime as smooth as a piece of polished stone; the red of the ochre rendering it of a rich brown color.

In less than a week the floors made in this way were sufficiently hard to bear the weight of a horse without indentation.

Roofs were made in the same manner without the coloring matter, which was added only to give the floors a better tint than the gray of the mortar. These roofs were perfectly water-proof, and were unaffected by sun or rain.

In the city of Monterey, sidewalks in the principal streets are made in the same manner, and some of them have lasted for years, wearing through like a block of stone.

The great durability and strength of these floors and roofs is entirely owing to the pounding operation above described, as the same materials were tried in the ordinary way without success.

The writer has not had occasion to make use of this process in this climate, but gives a description, hoping that it may be of value to others who may have occasion to lay floors of lime in architectural or engineering works. He has never heard of this method being employed in this country; although it seems singular that it should be used so generally by a neighboring nation, and be wholly unknown to our builders.

APPENDIX.

The following correspondence, growing out of the above, is here printed:

From Estevan A. Fuertes, C. E., Member of the Society.

A paper read before the Society, upon a Mexican method of consolidating Mortars, described as of extreme hardness, suggested to my mind that, perhaps, its author might be mistaken in attributing its main durability and hardness to the slow system of consolidating the road-bed and its cover.

My doubts have grown out of the circumstance that the author says (without seeming to attach much importance to the fact), that ochre, or a similar pigment, was mixed with the mortar.

I think that the coloring matter, believed to be of secondary importance, is the main ingredient which determines the superiority of the cement described; and instead of its being ochre, it was under-burnt brick dust.

If I am not the one who is mistaken, the consideration of this subject will bring up for discussion the method of obtaining a cheap and superior cement, that, I believe, has not been used much in this country.

The Hydraulic Engineer has much need of studying the causes which induce the "setting of mortars," because it is almost certain that the resistance of such materials as bricks, limes and cements, depends upon their conditions of crystallization.

I am aware of only two methods of hardening the silicates usually employed in hydraulic works viz., the gradual chemical change (crystallization in the slow humid way, as in submerged foundations, etc.), and the quick vitrification under the influence of intense heat, as employed in brick-making.

It is notorious that under-burnt brick resists very badly the influence of atmospheric wear, especially near salt water. I have crumbled in my hand an under-burnt brick, one year after its exposure to the sea spray, and gunpowder was manufactured from the nitrate of potassa formed upon its porous substance; but the same clay burned with chalk, making a double salt of carbonate of lime and silicate of alumina, or rather a sub-crystalline double silicate of lime and alumina, after being ground, made a cement susceptible of receiving a splendid lustre, and withstood the action of the spray and of the waves without apparent change.

Both the limestone and the brick had been used separately as building material in a burying ground near the sea shore, where the experiments were conducted in 1861. Eighteen months of exposure for the stone and twelve months for the brick, were sufficient to render both materials useless; but when burnt, ground and mixed they stood much better than the finest and distinctly crystalline marbles.

At the end of three years, or more than the sum of the times of durability of each material, I left the place where the experiments were made, and then

the cement had not changed where the surface had been left rough, nor tarnished where it had been polished.

A cement called "Revocado" by Spanish Engineers is made by mixing in several proportions fat limes with sand and under-burnt brick dust. The usual proportions are measured by equal volumes of the three materials; but when the cement is to be used for stopping roof leaks, cementing cellars, or where blows upon the cement are not anticipated, the proportion of sand is greatly diminished, and even suppressed altogether.

The Spanish learned the compounding of this cement from the Biscayans probably; and I doubt if the Romans had anything to do with its introduction in Spain, because the ruins of ancient water channels with "revocado" exist in the Basque provinces where neither Romans nor Moors ever penetrated. The Biscayans, in their turn, are the most ancient people with whom we are acquainted, it being probable that they preceded the Phœnicians.

I have seen Spanish "revocado" in Mexico, and it is natural to suppose that the Spaniards introduced the art in that country during the Conquest.

Now, may it not be possible that the ochre referred to by Gen. Ellis is only powdered brick, used to make the excellent and hard hydraulic "revocado?"

The description given of the method of consolidating the mortar, etc., and even the employment of wooden compressors, explain accurately the process still followed in Spanish countries to form the floorings of plazas, public walks, etc.

In many cases, immediately before the cement becomes set, its surface is polished with a smooth cobble stone until it acquires a high and lasting lustre.

From. GEN. ELLIS, in reply to the above:

Having read the remarks of Mr. Fuertes upon my recent paper relating to hard line floors, I apprehend that he did not give sufficient attention to the process therein described.

In no case was the red pigment mixed with the lime and sand, as he supposes, but was solely used for a surface coloring after the hardening process was completed. Roofs and sidewalks of equal hardness were also made by the same process of successive poundings without the coloring matter and finished by polishing in the same manner as the colored floors.

The pigment used upon the floors was not brick dust, but a red earth found in the vicinity, probably a fine clay colored with sesquioxide of iron. Bricks were not used in that part of the country; "adobes" taking their place in building.

It will thus be seen that the material of which the described floors and roofs were made was not the same as the "revocado" used in Spain and Southern Mexico, described by Mr. Fuertes. Is not the term *revocado* essentially the same in meaning as the more common Castilian word *revoque*, one being the participle and the other the noun corresponding to the Spanish verb *revocar*, the nearest English equivalent to which, in an engineering sense, is to *rough cast*. This implies an admixture of coarse material in the mortar. "Revoque" was known to the Romans as "*parietis linimentum*."

Mr. Fuertes is, I think, in error when he attributes any rapidity of setting, greater hardness when set, or improved hydraulic qualities, to the mixture of burnt or underburnt brick in any proportions with lime. The experiments of Smeaton show conclusively that the only gain is the slight amount of moisture that the brick will absorb from the lime and favor its *drying*.

The only way in which hydraulic properties can be given to a compound of silicate of alumina and carbonate of lime is by burning them together after being mixed, as in the production of artificial cement. This is exactly what was done in the case Mr. Fuertes recounts; clay and chalk were burned together, and if in proper proportions would form an excellent artificial hydraulic cement. It is not remarkable that neither should be a good building material by itself.

If the "revocado" of the Spanish possesses any quick setting or hydraulic qualities, it is probably not owing to the admixture of common brick, but to some qualities of the lime, or perhaps what Mr. Fuertes has taken to be brick was artificial "trass," formerly much used, which was burned like brick, and when added to mortar, gave it hydraulic properties.

I think it highly probable that the process of pounding ordinary lime mortar for many successive days in order to give it hardness, and afterwards polishing the surface, originally came from Spain to Mexico, and is probably an ancient practice. The only matter of surprise is that it has not become more generally known and used.

Note by the Printing Committee.

Is this "pounding process" of the Mexicans anything more than a simple yet effectual method of freeing the mortar of its *surplus water*, and thereby insuring a condition in which the lime can pass to a crystalline carbonate, at the same time compacting the whole mass into the best possible state?—*Vide* "Transactions" of this Society, No. X.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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AT A MEETING OF THE SOCIETY, MARCH 20TH, 1872.

NOTES ON THE RESISTANCE OF BRICKS TO A CRUSHING FORCE.

A paper presented by GEORGE S. GREENE, JR., C. E., Member of the Society.

The bricks were of the kind used in the construction of the South Gate-House of the New Reservoir in the city of New York, by Fairchild, Walker & Co., Contractors.

The experiments were made by Gen. George S. Greene, at Cornell & Co.'s, Centre Street, September 20th, 1860, in Hatfield's Hydraulic Press for testing building materials, built by R. Hoe & Co.

The bricks were what are known as hard brick, and manufactured at the yard of Wm. Call, Haverstraw, on the Hudson river; they are regarded as average samples of the mass of brick used in the construction of the Gate-House. The experiments were not made in the interest of any person, but solely to determine the actual strength of the brick. In order to bring them within the power of the machine, but little more than half of a brick was used. The pieces of brick were first dressed by a stone-cutter, and then ground down on a grind-stone. The faces exposed to pressure were not perfect planes, and therefore a

layer of wood and sand was interposed between the faces of the machine and those of the bricks.

Dimensions of the Brick used in Experiments, in Inches and Decimals.

	THICK.	WIDE.	BROAD.				
No. 1.—	2.30 x	3.52 x	4.40	15.488 sq.	in.	exposed to	pressure.
No. 2.—	2.24 x	3.50 x	4.46	15.610	"	"	"
No. 3.—	2.34 x	3.50 x	4.52	15.820	"	"	"
No. 4.—	2.34 x	3.46 x	4.46	15.4316	"	"	"
No. 5.—	2.30 x	3.46 x	4.50	15.570	"	"	"
No. 6.—	2.28 x	3.46 x	4.60	15.916	"	"	"

No. 1.—At 30,000 lbs. (= 1,937 lbs. per sq. in.) cracked in centre; kept at 50,000 (= 3,228.3 lbs. per sq. in.) without crushing. Brick between two pieces of board, half an inch thick.

No. 2.—Had a layer of sand. Sign of crack at 50,000 lbs. (= 3,203 lbs. per sq. in.); kept at 52,500 (= 3,362.2 per sq. in.) for three minutes, but did not crush. Crack did not extend through brick, nor was it broken into two parts.

No. 3.—Crushed to pieces at 43,500 lbs. (= 2,749.7 lbs. per sq. in.); packed with sand.

No. 4. Packed with two pieces of cigar-box wood; edges crushed off at 30,000 lbs. (= 1,994.1 lbs. per sq. in.)

No. 5.—Packed with sand; cracked at 27,000 lbs. (= 1,734.1 lbs. per sq. in.); crushed at 32,000 lbs. (= 2,055.3 lbs. per sq. in.) Brick crushed and cracked in all directions; did not fall to pieces as did No. 3.

No. 6.—Packed in sand; commenced to crack at 30,000 lbs. (= 1,884.9 lbs. per sq. in.); crushed to pieces at 46,500 lbs. (= 2,921.6 lbs. per sq. in.)

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AT A MEETING OF THE SOCIETY, JANUARY 8TH, 1873.

EXPERIMENTS ON THE RESISTANCE OF STONES TO CRUSHING.

A paper presented by C. B. RICHARDS, M. E., Member of the Society.

The accompanying tables present the data and results of experiments made by direction of General Franklin, at the Colt Company's Armory, to ascertain the relative resistance of various American building stones to crushing.

The specimens were furnished by Mr. J. G. Batterson, and were shaped with great accuracy at his marble works in this city. They were selected from old and dry stones of the best quality of their kinds, and were worked into nearly perfect cubes with very flat and smooth faces. Two different sizes of cubes were tested, their edges measuring very nearly one inch and one and a half inch respectively.

The testing machine used in the experiments is one designed by the writer for the Colt Company, who had it constructed for use in their Armory, both for testing materials used in their own manufactures, and also for making tests for the engineering public. As a description of this machine was published in the "Scientific American," of March 16th, 1872, and in several other mechanical periodicals, it is unnecessary

to describe it here; it is only desirable to state that the apparatus for weighing the strains consists of a very sensitive platform scale of fifty tons capacity, and that the machine was built after a long experience with two smaller, similar machines, one of which was constructed by the writer as early as 1867. Experience with this large machine in operating on hundreds of specimens, has proved it to be admirably adapted for accurate research, and leaves no room for doubt as to the correctness of its indications.

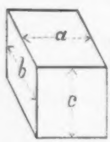
The fixtures between which the stones were crushed consist of hardened steel hemispheres, with their curved surfaces seated in corresponding cavities made in steel blocks, which are fixed in the testing machine. These hemispheres can rock in every direction, and their flat surfaces, therefore, accommodate themselves to the direction of the surfaces of the specimens.

Single thicknesses of thin "lace" leather were applied between the steel and stone surfaces to ensure uniform distribution of the pressure upon the specimen.

The pressure was in all cases applied to those surfaces of the cubes which were parallel with the natural bed of the stone.

Table **1** gives the description and dimensions of each specimen, the strain required to crush it, and the resistance per square inch deduced from the area crushed and the crushing strain.

The sides of the cubes and their dimensions referred to in the table, columns 3, 4, 5 and 6, are indicated by letters, as shown in this diagram.



The side on which the pressure was applied is shown in each case in the table by the letter in the sixth column. The area of the surface on which the pressure was given, will there be obtained from two of the three dimensions given in the third, fourth and fifth columns, as follows:

$$\text{If the pressure was exerted on } \left\{ \begin{matrix} a \\ b \\ c \end{matrix} \right\} \text{ the area} = \left\{ \begin{matrix} a \times b \\ b \times c \\ c \times a \end{matrix} \right\}$$

Table **2** exhibits a resumé of the results of all the tests, grouped according to the kind of stone and size of specimen; giving the minimum, average and maximum resistance per square inch of all the specimens of each kind.

All the specimens failed by breaking up into slender prisms and pyramids whose axes were normal to the surfaces on which the pressure

was exerted. Specimen No. 8, (Table **1**), split into four parts with a strain of 6,250 lbs., and afterwards sustained an additional load of 5,210 lbs., without other indications of failure.

Several other specimens split into two or four nearly equal parts, with loads somewhat less than those which ultimately produced crushing.

In all cases the strain was gradually applied, and increased to the crushing point by pouring shot into the apparatus by which the pressure is produced.

It will be noticed that several of the specimens broke with strains of even thousands of pounds. This coincidence probably arises from the fact that it requires some seconds of time to move the large weight on the weigh-beam of the testing machine, and this needs to be moved for every two thousand pounds increment of strain, (the intermediate pressures being otherwise indicated at the beam). While this change is being made, the strain then exerted is maintained unaltered, while at other times the strain is continually though slowly increasing. It is reasonable to suppose that if the specimen is slowly giving way while the adjustment of the weight is being made, opportunity is then given for it to yield and fail at the pressure then on, which will be at some certain thousand of pounds; but if the strain had been slowly increasing during the time while the specimen was yielding, the indication of the machine would have been a few pounds higher.

HARTFORD, CT., January, 1873.

TABLE 1.

No.	KIND OF STONE.	Dimensions. A	Dimensions. B	Dimensions. C	Side on which the press's was applied.	Pressure at which crushing occurred.	Resistance to crushing per square inch.
		in	in.	in.		lbs.	lbs.
1	White Marble from Canaan, Ct.........	0.980	0,990	0.990	*a*	4,810	4,958
2	do. do.	1.000	0.963	0.980	*b*	6,280	6,654
3	do. do.	0.990	0.998	0.963	*a*	5,000	5,060
4	do. do.	0.997	0.990	0.992	*a*	6,500	6,585
5	White Marble from Alford, Mass.......	1.000	0.998	1.005	*a*	5,000	5,010
6	do. do.	1.002	1.005	1.004	*b*	3,940	3,905
7	do. do.	1.000	1.005	1.000	*a*	5,990	5,960
8	Blueish Marble from Lee, Mass.........	0.995	0.999	0.998	*a*	11,460	11,528
9	do. do.	0.984	0.998	1.000	*b*	7,690	7,705
10	Granite, from Westerly, R. I............	1.000	1.008	1.004	*b*	19,000	18,778
11	do. do.	1.002	1.005	1.000	*a*	15,700	15,590
12	Granite, from Plymouth, Ct............	1.000	0.993	0.995	*a*	8,560	8,620
13	do. do.	0.997	0.995	0.997	*c*	10.350	10,412
14	Granite, from Concord, N. H............	1.000	1.000	0.985	*c*	8,680	8,812
15	do. do.	1.002	0.983	1.003	*a*	9,690	9,838
16	White Marble, from Alford, Mass.......	1.00[illegible]	1.004	0.997	*a*	6,000	5,976
17	Granite, from Quincy, Mass.............	1.001	1.001	1.004	*c*	15.700	15,622
18	do. do.	1.005	0.996	1.001	*r*	13,000	12,923
19	do. do.	1,001	0.997	1.003	*b*	11,730	11,730
20	Granite, from Fox Island, Maine.	1,000	1.004	0.987	*c*	14.000	14,185
21	do. do.	1.000	1.001	1.005	*c*	12,360	12,299
22	do. do.	1.007	1.000	1.005	*b*	12,900	12,836
23	do. do.	0.995	1.004	1.002	*a*	11,880	11,892
24	Granite, from Jonesboro, Maine........	0.990	1.007	1.006	*b*	11,380	11,233
25	do. do.	1.007	0.994	1.005	*a*	12,320	12,307
26	do. do.	0.996	1.005	0.998	*b*	16,000	15,952
27	Sandstone, from Amherst, Ohio.........	1.002	0.988	0.995	*c*	7,740	7,463
28	do. do.	0.985	0.998	1.000	*c*	8,350	8,477
29	do. do.	0.992	0.991	0.999	*c*	8.050	8,123
30	do. do.	0,988	0.996	1.009	*b*	9.000	8,955
31	do. do.	0,996	1.002	1.004	*b*	6,180	6,141
32	Sandstone, from Portland, Ct...........	0,993	1.063	1.006	*c*	5,800	5,806
33	do. do.	1,002	0,985	0.993	*a*	6,240	6,322
34	do. do.	1.004	0.097	0.996	*a*	8,260	8,252
35	do. do.	1.001	1.001	1.001	*a*	10,950	10,928
36	Sandstone, from Nova Scotia...........	0,993	1.006	0.998	*c*	10,220	10,322
37	do. do.	0,999	0,997	1.005	*a*	7,130	7,158
38	do. do.	1,011	0.997	0.998	*b*	6,500	6,532
39	do. do.	1.005	0.998	0.998	*a*	9,220	9,193
40	do. do.	1.005	0.999	0.996	*b*	9,320	9,366
41	White Statuary Marble, Carrara, Italy..	1.494	1.487	1.493	*a*	21,600	9,723
42	White Marble, from Canaan, Ct........	1.476	1.483	1.493	*a*	19,250	8,794
43	do. do.	1.487	1.479	1.484	*a*	17,140	7,793
44	Sandstone, from Windsor, Ct...........	1.489	1.488	1.484	*a*	22,000	9,929
45	do. do.	1.495	1.484	1.488	*a*	27,850	12,553
46	White Marble, from Derby, Ct..........	1.489	1.486	1.466	*b*	21,300	9,777
47	do. do.	1.486	1.478	1.490	*a*	18,900	8,605
48	Blueish Marble, from Lee, Mass.........	1.475	1.485	1.480	*c*	30,460	13,953
49	do. do.	1.480	1.476	1.483	*b*	39,300	17,954
50	White Marble, from Lee, Mass..........	0.993	0.998	0.988	*a*	12,800	12,916
51	do. do.	0.992	0.990	0.983	*b*	13,900	13,972

TABLE 2.

KIND OF STONE.	LOCALITY OF QUARRY.	Number of Specimens crushed of each kind.	Nominal size of Specimens.	Resistance to Crushing per Square Inch.		
				Minimum.	Average.	Maximum.
			in.	lbs.	lbs.	lbs.
White Marble......	Canaan, Ct.	4	1	4,958	5,812	6,585
do.	do.	2	1½	7,794	8,294	8,794
do.	Alford, Mass.........	4	1	3,905	5,213	5,976
do.	Lee, Mass...........	2	1	12,917	13,444	13,972
Blueish Marble	do.	2	1	7,705	9,616	11,528
do.	do.	2	1½	13,953	15,953	17,954
Granite...........	Westerly, R. I.	2	1	15,59[illegible]	17,184	18,778
do.	Plymouth, Ct........	2	1	8,620	9,516	10,412
do.	Concord, N. H.......	2	1	8,812	9,325	9,838
do.	Quincy, Mass........	3	1	11,730	13,425	15,622
do.	Fox Island, Maine...	4	1	11,892	12,803	14,185
do.	Jonesboro, Maine.....	3	1	11,234	13,164	15,952
Sandstone	Portland, Ct.........	4	1	5,806	7,826	10,928
do.	Amherst, Ohio.......	5	1	6,141	7,832	8,956
do.	Nova Scotia.........	5	1	6,532	8,512	10,322
do.	Windsor, Ct.........	2	1½	9,930	11,241	12,553
Statuary Marble ...	Carrara, Italy.......	1	1½	.....	9,723	.....

APPENDIX.

The following additional communication from Mr. Richards, in reply to some enquiries, is here printed:

With reference to these experiments on stones, one object in having the specimens made of two different sizes, was to attempt to ascertain whether the size would affect the modulus of rupture. It was anticipated that the larger cubes would give the higher results, and this seems to be the fact. But, in the communication, attention was not called to this circumstance, because it can hardly be supposed that reliable average results can be obtained from a test of only two specimens, and there were no more than two of the large specimens of any one kind of stone. This doubt would apply particularly to cases where the extreme results vary so greatly as those in question. It is to be noticed in the tables that in some cases the minimum modulus for the large cubes is smaller than the maximum for the small ones of the same kind of stone.

It would be interesting to learn where the limit is, when an increase of the size of the specimen ceases to affect the modulus derived from

the experiments. But I imagine that this limit would vary with the different kinds of stone, and that the solution of the question would require a testing apparatus of considerable capacity. That the relative proportions of the specimens be made to correspond with those of stones commonly used in practical work, is undoubtedly important, and ought to be carried out. It seems probable, however, that even in this case, the thickness of the experimental piece ought to be considerable, say at least one and a half inches for some kinds of stone, and this would involve a large area to be crushed, and consequently require the application of great strains.

The more any one makes test of the strength of materials, the more he realizes the importance of being able to test large samples, and of having powerful testing machines, such as Kirkaldy's.

Since this communication was prepared, my attention has been called to experiments made by the Commission of 1851, on marbles, &c., for the United States Capitol Extension, (see Architect's report, accompanying the President's message, 32d Congress, 2d Session, 1852).

This Commission discovered the somewhat remarkable fact that the effect of pieces of sheet lead placed between the stone samples and the steel surfaces of the testing apparatus, was to occasion the failure of the specimen with about half the load it would sustain if pressed directly by the steel surfaces; that is, with the steel and stone surfaces in contact.

Now, in my experiments, thin leather was used between the stone and steel surfaces, as stated. This leather must have become practically rigid under the pressure it sustained, as seems to be proved by the way the specimens broke up into numerous slender pyramids, and by the friable character of the whole substance of the fractured parts, which can in most instances be crushed to powder between the fingers. These conditions of the broken pieces seem to indicate that the pressure was applied in a way that caused all parts of the specimens to fail at the same time, which must be the case when steel surfaces are applied fairly to the stone. With lead interposed, the circumstances are different, because this metal retains its softness at such pressures as those in question.

LVII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

AT A REGULAR MEETING OF THE SOCIETY, JANUARY 17TH, 1872.

QUAY AND OTHER RETAINING WALLS.

A paper read by JOHN D. VAN BUREN, JR., C. E., Member of the Society.

I.

The problem presented for solution in this paper is to establish practical formulæ for the dimensions of Quay and other Retaining Walls of various shapes and under various conditions.

In designing a quay retaining wall, the practical questions must precede the theoretical ones. The data required for the solution of the latter, and which depend upon the former, are :

1°. The depth of water required along the quay, and the total height of the wall ;

2°. The face profile or batter of the wall,—which, while answering the requirements of a convenient quay, must be such as to secure economy in its section ;

3°. The back profile,—which, while not necessitating too broad a base, (and, consequently too extensive a foundation) must be such as to utilize as far as possible the vertical pressure of the earth filling ;

4°. The character of the earth filling ; which should be considered the worst that is likely to occur,—especially because a quay-wall is in a state of constant vibration, and the friction of the earth filling is thereby reduced from that of quiescence to nearly that of motion ;

5°. The character of the material to be used in the construction of the wall;

6°. The degree of stability required, both in the wall and the foundation.

Quay-walls being immersed in water present some difficulties in the way of a mathematical solution of the question. For there are two additional pressures to be considered: a pressure of water favorable to stability against the face, and a pressure of water unfavorable to stability upward against the base.

In the following investigations, then, the wall will be considered as acted upon by four forces: 1°. The weight of the wall, and the weight of the mass of earth overlying the offsets, if any there are; 2°. the pressure of the earth behind the wall, *supposing it to be saturated with water;* 3°. the pressure of the water on the face of the wall; 4°. the pressure of the water on the base of the wall.

The pressure of the earth will be supposed to act against the plane B D (fig. 1), having the inclination of the back of the lowest offset; and the weight of the earth between this plane and the wall will be combined with that of the wall.

The batter or profile of the face will be assumed, and also the profile of the back; the problem will then be to find the top width a', or the base b.

Formulæ for the pressure of earth have already been established upon Coulomb's principle of the wedge of maximum pressure, by various distinguished authors, including Poncelét, Moseley, and others. We shall base the following formulæ upon these.

The general case is given in Moseley's Mechanics of Engineering, (edited by Prof. Mahan), p. 424, eq. 443.

[NOTE.—The worst conditions under which a wall, with its base exposed to a water pressure, can be placed, are, when the earth behind it is saturated with water, *and the depth of water is equal to the breadth of the base.* For supposing the earth to be saturated with water, the tendency of the water alone to overturn the wall will then be greatest; *i. e.*, the difference of moments of water pressure on the base and face will then be a maximum.

Let b = base of wall; d = depth of water; $u_1 = 1$.

This difference can then be represented by

$$\frac{b^2 d}{2} - \frac{d^3}{6} = u \quad \text{. . . . . (a)}$$

Placing the first differential coefficient of u in respect to $d = 0$ and solving for d:

$$\frac{d\,u}{d\,.\,d} = \frac{b^2}{2} - \frac{3\,d^2}{6} = 0$$

$$\therefore \frac{b^2}{2} = \frac{3\,d^2}{6}$$

$$\text{or, } d = b, \text{ a maximum.} \quad \text{. . . . . (b)}$$

When the base is not exposed to a water pressure, evidently, *low water* is the worst stage of the tide.]

$P =$ total pressure of earth against an unit of length of an inclined plane

$$= \tfrac{1}{2} \cdot \frac{u_3 x^2 \sin \varphi}{\cos^2(\beta + \varphi + \varphi_2)} \cdot \left\{ \left(\frac{\cos(\beta + \varphi_2)}{\sin \varphi} \right)^{\frac{1}{2}} - \left(\frac{\sin(\varphi + \varphi_2)}{\cos \beta} \right)^{\frac{1}{2}} \right\}^2 \ldots\ldots (1).$$

the top surface being then level:

Where, $u_3 =$ weight per cubic foot of earth;

$\varphi =$ angle of repose of earth;

$\varphi_2 =$ angle of friction or repose of earth against the plane B D, or back of wall;

$h = x =$ vertical height of wall;

$\beta =$ angle of batter of back of wall measured towards it, *i. e.*, to right, from the vertical (fig. 1);

φ_2 may be neglected,—its omission being slightly in favor of stability.

Making $\varphi_2 = 0$ in (1), P becomes:

$$P = \tfrac{1}{2} \cdot \frac{u_3 x^2 \sin \varphi}{\cos^2(\beta + \varphi)} \cdot \left\{ \left(\frac{\cos \beta}{\sin \varphi} \right)^{\frac{1}{2}} - \left(\frac{\sin \varphi}{\cos \beta} \right)^{\frac{1}{2}} \right\}^2 = \tfrac{1}{2} x^2 u_3$$

$$\cdot \frac{(\cos \beta - \sin \varphi)^2}{\cos^2(\beta + \varphi) \cos \beta} \ldots\ldots (2).$$

If $\beta = 0$, the back of the wall is vertical, and:

$$P = \tfrac{1}{2} x^2 u_3 \cdot \frac{1 - \sin \varphi}{1 + \sin \varphi} \ldots\ldots (3).$$

If $\varphi = 0$, and $\beta = 0$, the back is vertical and earth perfectly liquid,

$$P = \tfrac{1}{2} x^2 u_3 \ldots\ldots (4).$$

By placing in (2) $u_4 = u_3 \cdot \dfrac{(\cos \beta - \sin \varphi)^2}{\cos^2(\beta + \varphi)}$, $P = \dfrac{u_4}{2} \cdot x^2 \sec \beta \ldots\ldots (5)$,

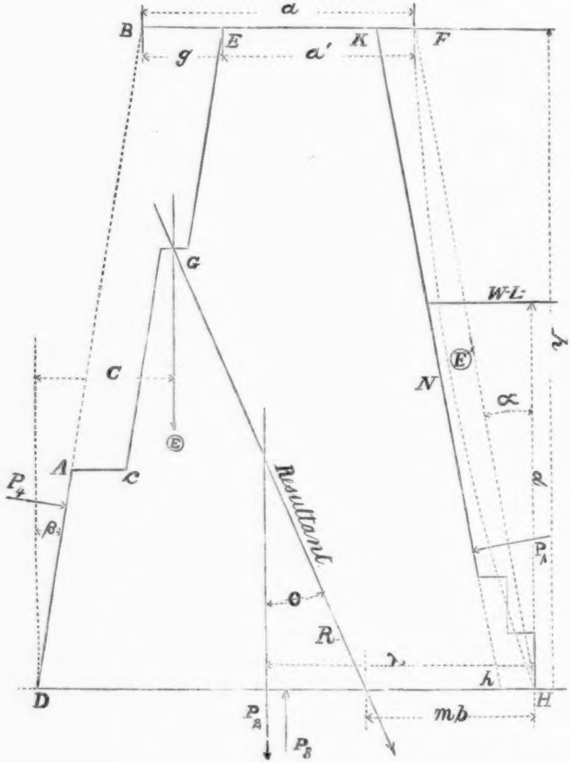
a formula identical in form with that for a perfect liquid.

u_4, then, will have one of the five following values:

1. General case, $\quad u_4 = u_3 \cdot \dfrac{(\cos \beta - \sin \varphi)^2}{\cos^2(\beta + \varphi)}$;
2. Back vertical, $\quad u_4 = u_3 \cdot \dfrac{1 - \sin \varphi}{1 + \sin \varphi}$;
3. Back inclining backward at angle, $\quad -\beta, \; u_4 = u_3 \cdot \dfrac{(\cos \beta - \sin \varphi)^2}{\cos^2(\varphi - \beta)}$;
4. Back inclining backward at angle, $\quad 90^\circ - \beta = \varphi, \; u_4 = 0$;
5. Earth perfectly liquid, $\quad \varphi = 0, \; u_4 = u_3$.

$\ldots\ldots (6).$

Fig. 1.



In the following investigations the top surface of the earth is supposed to be level, as they relate more particularly to Quay-walls.

An unit only of length of wall is considered

Notation:

O = origin of moments = point where resultant R cuts base $H\ D$;

P_1 = water pressure on face of wall;

P_3 = water pressure on base of wall:

P_2 = weight of wall + weight of mass of earth $A\ B\ E\ G\ C\ A$;

P_4 = pressure of earth filling against plane $B\ D$;

α = angle of face of wall or line HF, with vertical, measured around to left;

β = angle of back of wall or line BD, with vertical measured around to right;

u_1 = weight of one cubic foot of water;

u_2 = weight of one cubic foot of wall;

u_3 = weight of one cubic foot of earth filling;

φ = angle of repose of earth filling;

$u_4 = u_3 \cdot \dfrac{(\cos\beta - \sin\varphi)^2}{\cos^2(\beta+\varphi)}$, and $P = \dfrac{u_3 h^2}{2} \cdot \dfrac{(\cos\beta - \sin\varphi)^2}{\cos^2(\beta+\varphi)\cos\beta} = \dfrac{u_4 h^2}{2} \cdot \sec\beta$;

E = difference of weight of stone prism $A\,B\,E\,G\,C\,A$, and earth prism of same volume = Area $A\,B\,E\,G\,C\,A \times (u_2 - u_3) = (u_2 - u_3)\,S$;

E^1 = weight of volume $F\,H\,K$ of wall = excess of weight of volume $D\,A\,C\,G\,E\,F\,H$ over weight of volume of wall;

λ = distance from H to line of weight of wall *and* earth $A\,B\,E\,G\,C\,A$, *i. e.*, to vertical line through the centre of gravity of wall and earth $A\,B\,E\,G\,C\,A$;

p = arm of Moment $P\,p$;

a = width from F, to the point B where the plane BD cuts the horizontal FE;

a^1 = top width of wall = EK; b = base of wall; d = depth of water;

θ = angle of resultant force R, with vertical;

$\pm C$ = distance measured horizontally of centre of gravity of $A\,B\,E\,G\,C\,A$, or E, from D; + towards H;

C^1 = distance measured horizontally of centre of gravity of E^1 from H;

$m\,b = m\left(a + h(\tan\alpha + \tan\beta)\right)$ = distance from H to O, where resultant R cuts the base HD; m = a constant coefficient.

Taking the moments about O,

$$P_1 p_1 = \frac{u_1 d^2}{2}\sec\alpha\left[\frac{d}{3}\sec\alpha - m\left(a + h(\tan\alpha + \tan\beta)\right)\sin\alpha\right] = \frac{u_1 d^2}{6}$$

$$\cdot\left[d(1 + \tan^2\alpha) - 3\,m\left(a + h(\tan\alpha + \tan\beta)\right)\tan\alpha\right];$$

$$P_2 p_2 = \left(\lambda - m\,(a + h\,(\tan\alpha + \tan\beta)\right)\left[\frac{2\,a + h\,(\tan\alpha + \tan\beta)}{2} \cdot h\,u_2 - E - E^1\right];$$

$$P_3 p_3 = -\frac{u^1 d}{2} \cdot (1 - 2\,m)\left(a + h\,(\tan\alpha + \tan\beta)\right)^2;$$

$$P_4 p_4 = -\frac{u_4 h^2}{6}\left[h\,(1 + \tan^2\beta) - 3\,(1 - m)\left(a + h\,(\tan\alpha + \tan\beta)\right)\tan\beta\right];$$

$$\left\{\begin{aligned} \lambda = & \frac{u_2\, h\, [3\, a \left(a + h\, (2 \tan \alpha + \tan \beta)\right) + h^2\, (\tan \alpha + \tan \beta)\, (2 \tan \alpha + \tan \beta)]}{3\, u_2\, h \left(2\, a + h\, (\tan \alpha + \tan \beta)\right) - 6\, E - 6\, E^1} \\ & \frac{-6 \left(a + h\, (\tan \alpha + \tan \beta) - c\right) E - 6\, c^1\, E^1}{3\, u_2\, h \left(2\, a + h\, (\tan \alpha + \tan \beta)\right) - 6\, E - 6\, E^1}. \end{aligned}\right.$$

Now, for the sake of brevity place $2 \tan \alpha + \tan \beta = p$, and $\tan \alpha + \tan \beta = q$. Hence :

$$P_1\, p_1 = \frac{u_1\, d^2}{6} \cdot [d\, (1 + \tan^2 \alpha) - 3\, m\, (a + h\, q) \tan \alpha];$$

$$P_2\, p_2 = \left(\lambda - m\, (a + h\, q)\right) \left[\frac{2\, a + h\, q}{2} \cdot h\, u_2 - E - E^1\right];$$

$$P_3\, p_3 = -\frac{u_1\, d}{2} \cdot (1 - 2\, m)\, (a + h\, q)^2;$$

$$P_4\, p_4 = -\frac{u_4\, h^2}{6} \cdot [h\, (1 + \tan^2 \beta) - 3\, (1 - m)\, (a + h\, q) \tan \beta];$$

$$\lambda = \frac{u_2\, h\, [3\, a\, (a + h\, p) + h^2\, p\, q] - 6\, (a + h\, q - c)\, E - 6\, c^1\, E^1}{3\, u_2\, h\, (2\, a + h\, q) - 6\, E - 6\, E^1.};$$

and since the moments about O are in equilibrium :

$$\begin{aligned} & u^1\, d^2\, [d\, (1 + \tan^2 \alpha) - 3\, m\, (a + h\, q) \tan \alpha] + u_2\, h\, [3\, a\, (a + p\, h) + p\, q\, h^2] \\ & -6\, (a + q\, h - c)\, E - m\, (a + q\, h)\, (3\, u_2\, h\, (2\, a + q\, h) - 6\, E - 6\, E^1) - 6\, c^1\, E^1 \\ & = 3\, u_1\, d\, (1 - 2\, m)\, (a + q\, h)^2 + u_4\, h^2\, [h\, (1 + \tan^2 \beta) - 3\, (1 - m)\, (a + q\, h) \tan \beta] \ldots (7. \end{aligned}$$

Reducing,—and collecting in order as they come all terms involving a into the first member :

$$\begin{aligned} & -3\, u^1\, m\, d^2 \tan \alpha\, a + 3\, u_2\, h\, a^2 + 3\, u_2\, h^2\, p\, a - 6\, E\, a - 6\, u_2\, m\, h\, a^2 - 3\, m\, u_2\, h^2\, q \\ & + 6\, m E^1 a + 6\, m E a - 3\, u_1 (1 - 2\, m) d\, a^2 - 6\, u_1 (1 - 2\, m) d\, h\, q\, a + 3\, u_4 (1 - m) \tan \beta\, h^2 \\ & = 3\, u_1\, (1 - 2\, m)\, d\, h^2\, q^2 + u_4\, h^2\, [h\, (1 + \tan^2 \beta) - 3\, (1 - m)\, h\, q \tan \beta] \\ & -u_1\, d^2\, [d\, (1 + \tan^2 \alpha) - 3\, m\, h\, q \tan \alpha] - u_2\, h^3\, p\, q + 6\, (h\, q - c)\, E + 3\, m\, u_2\, h^3\, q^2 \\ & -6\, m\, h\, q\, E + 6\, c^1\, E^1 - 6\, m\, q\, h\, E^1 \ldots\ldots\ldots\ldots\ldots (8) \end{aligned}$$

or :

$$\begin{aligned} & 3\, a^2 \left(u_2\, h\, (1 - 2\, m) - u_1\, (1 - 2\, m)\, d\right) + a\, [3\, h^2 \left(u_2\, (p - 3\, m\, q) + u_4\, (1 - m) \tan \beta\right) \\ & -6\, u_1\, (1 - 2\, m)\, q\, d\, h - 3\, u_1\, m\, d^2 \tan \alpha - 6\, (1 - m)\, E + 6\, m\, E^1] \\ & = h^3\, [u_4 (1 + \tan^2 \beta) - 3 (1 - m) u_4\, q \tan \beta + 3\, m\, u_2\, q^2 - u_2\, p\, q] + 3\, (1 - 2\, m) h^2 u_1 d\, q^2 \\ & + h [6\, q E - 6\, m\, q\, E - 6\, m\, q\, E^1 + 3\, m\, q\, u_1 d^2 \tan \alpha] - u_1 d^3 (1 + \tan^2 \alpha) - 6\, c\, E + 6\, c^1 E^1 \\ & = h^3\, [u_4 (1 + \tan^2 \beta) - 3 (1 - m) q\, u_4 \tan \beta + 3\, m\, q^2\, u_2 - p\, q\, u_2] + 3 (1 - 2\, m)\, h^2 u_1 d\, q^2 \\ & + h\, [6\, (1 - m)\, q\, E - 6\, m\, q\, E^1 + 3\, m\, q\, u_1\, d^2 \tan \alpha] - u_1\, d^3\, (1 + \tan^2 \alpha) - 6\, c\, E \\ & + 6\, c^1\, E^1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9.) \end{aligned}$$

and we have (10), [Appendix].

where A = first term of second member, or the whole quantity outside the radical sign: and $\qquad a^1=a-(B\,E+F\,K)=a-(g+k)$ ....(10').

2°. Wall with equal batters, face and back, with offsets; *immersed.*

In this case, $\qquad \alpha=\beta;\ p=3\tan\alpha;\ q=2\tan\alpha;$

and we have (11), [Appendix].

2°, *a*. If in (11) E and $E^1=O$; we have the case of a trapezoidal section.

3°. Wall with face and back parallel, sloping back at angle α; *immersed.*

In this case, $\qquad \beta=-\alpha;\quad q=0;\quad p=\tan\alpha;$

and we have (12), [Appendix].

$$u_4=u_3\,\frac{(\cos\alpha-\sin\varphi)^2}{\cos^2(\varphi-\alpha)}.$$

If $90^\circ-\beta=\varphi$; $\quad u_4=0$, as it should.

3°, *a*. If in (12) E and $E^1=O$, we have the case of a battered wall, without offsets, with face and back parallel.

4°. Wall with vertical back and battered face with offsets, *immersed.*

In this case, $\beta=0$; and $u_4=u_3\,\dfrac{1-\sin\varphi}{1+\sin\varphi}$ (as in Rankines' Civ. Eng., p. 324, eq. 19), and we have (13), [Appendix].

4°, *a*. If in (13) E and $E^1=0$; the wall is without offsets, the back is vertical and the face battered.

5°. Wall with rectangular section.

In this case, $\qquad \alpha=0;\quad \beta=0;$

$$u_4=u_3\,\frac{1-\sin\varphi}{1+\sin\varphi};\ \text{and}$$

$$a=\frac{2\,(1-m)\,E-2\,m\,E^1}{2\,(1-2\,m)\,(u_2\,h-u_1\,d)}+\sqrt{\frac{h^3\,u_4-u_1\,d^3-6\,c\,E+6\,c^1\,E^1}{3\,(1-2\,m)\,(u_2\,h-u_1\,d)}+A^2}\ \ldots\ldots(14);$$

If E and E^1 in (14) are made $=0$, we have:

$$u_4=u_3\,\frac{1-\sin\varphi}{1+\sin\varphi},\ \text{and}$$

$$a=\sqrt{\frac{h^3\,u_4-u_1\,d^3}{3\,(1-2\,m)\,(u_2\,h-u_1\,d)}}\ \ldots\ldots\ldots\ldots(14a);$$

this is readily proven to be correct by making a special investigation.

II. Walls not Immersed.

In the following cases the walls are not immersed, and $u_1 = 0$;

1°. General case (eq. 10),

$$u_4 = u_3 \frac{(\cos \beta - \sin \varphi)^2}{\cos^2(\beta + \varphi)};$$

and we have (10*a*), [Appendix].

2°. Wall with equal batters, face and back, with offsets:

$$\beta = \alpha; \quad u_1 = 0;$$

and we have (11*a*), [Appendix].

$$u_4 = u_3 \frac{(\cos \alpha - \sin \varphi)^2}{\cos^2(\alpha + \varphi)}.$$

2°, *a*. If E and E^1 in (11*a*)=0, we have the case of a symmetrical trapezoidal section, and $u_4 =$, as before, and, consequently, (11*b*), [Appendix].

3°. Wall with face and back parallel, sloping back at angle α; with offsets.

In this case, $\beta = -\alpha; \quad u_1 = 0;$

and (12) becomes (12*a*), [Appendix.]

$$u_4 = u_3 \frac{(\cos \alpha - \sin \varphi)^2}{\cos^2(\varphi - \alpha)}$$

3°, *a*. If in (12*a*) E and $E^1 = 0$, we have a wall with face and back parallel sloping back at angle α, without offsets:

$$u_4 = u_3 \frac{(\cos \alpha - \sin \varphi)^2}{\cos^2(\varphi - \alpha)};$$

$$a = -\frac{h \tan \alpha \, [u_2 - u_4 \, (1-m)]}{2 \, u_2 \, (1-2 \, m)} + \sqrt{\frac{h^2 \, u_4 \, (1 + \tan^2 \alpha)}{3 \, u_2 \, (1-2 \, m)} + A_2} \ldots\ldots\ldots\ldots (12b)$$

4°. Wall with vertical back and battered face, with offsets.

$$\beta = 0; \quad u_1 = 0;$$

(eq. 13) becomes (13*a*), [Appendix].

$$u_4 = u_3 \frac{1 - \sin \varphi}{1 + \sin \varphi}.$$

4°, *a*. If in (13*a*) $E=0$, we have the wall without offsets:

$$u_4 = u_3 \frac{1 - \sin \varphi}{1 + \sin \varphi};$$

and, consequently, (13*b*), [Appendix].

5°. Rectangular section with offsets :

$$\alpha=0\,;\quad \beta=0\,;\quad u_1=0\,;$$

$$u_4=u_3\,\frac{(1-\sin\varphi)}{(1+\sin\varphi)}\,;$$

$$a=\frac{(1-m)\,E-m\,E}{u_2\,h\,(1-2\,m)}+\sqrt{\frac{h^3\,u_4-6\,c\,E+6\,c^1\,E^1}{3\,u_2\,h\,(1-2\,m)}+A^2}\,\ldots\ldots(14'a).$$

5°, a. If in (14'a) E and $E^1=0$; we have the wall without offsets :

$$a=h\sqrt{\frac{u_4}{3\,u_2\,(1-2\,m)}}=h\sqrt{\frac{u_4}{6\,u_2\,q^1}}\ldots\ldots\ldots\ldots(14b).$$

Where $$q^1=\frac{1-2\,m}{2},$$

which is identical with eq. 2, page 405, Rankine's Civil Engineering.

III.

The next step is to review the wall as computed, find the resultant force R and its angle Θ, *and check the computations by summing the moments.* To find R, we have only to resolve the component forces, P_1, P_2, P_3, and P_4, in respect to two rectangular co-ordinate axes, and determine the resultant moment by the usual methods of mechanics.

Taking the origin at H; the vertical through H for the axis of Y; and the horizontal through the same point for the axis of X, we shall have :

$$\begin{aligned}\Sigma\,P\cos\alpha=X&=P_1\cos\alpha_1+P_2\cos\alpha_2+P_3\cos\alpha_3+P_4\cos\alpha_4\\&=P_1\cos\alpha+P_2\cos 90^\circ+P_3\cos 90^\circ-P_4\cos\beta\\&=P_1\cos\alpha-P_4\cos\beta\ldots\ldots\ldots\ldots\ldots\ldots(15).\end{aligned}$$

$$\begin{aligned}\Sigma\,P\sin\alpha=Y&=P_1\sin\alpha_1+P_2\sin\alpha_2+P_3\sin\alpha_3+P_4\sin\alpha_4\\&=P_1\sin\alpha+P_2\sin 90^\circ-P_3\sin 90^\circ+P_4\sin\beta\\&=P_1\sin\alpha+P_2-P_3+P_4\sin\beta\ldots\ldots\ldots\ldots\ldots(16),\end{aligned}$$

where α is measured to the right *over* from axis of X; and β from left over from axis X. If the back of the wall slopes back, as in the case of a leaning wall, β is evidently *negative ;* and if the face of the wall should lean forward, α would also be *negative.* Otherwise they are both to be taken with the positive sign.

$$R = \sqrt{X^2 + Y^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17).$$

The resultant moment $R\,r$ (where r=its arm)

$$= P_1\,p_1 + P_2\,p_2 + P_3\,p_3 + P_4\,p_4 \quad \ldots\ldots\ldots\ldots\ldots (18);$$

where the signs + and − are implied in the terms; and the origin of moments is taken at 0.

Tendency to cause rotation from right over to left has the positive sign; $P_1\,p_1$ and $P_2\,p_2$ will, therefore, be *positive;* and $P_3\,p_3$ and $P_4\,p_4$, *negative.*

$$\text{Tan}\ \Theta = \frac{X}{Y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18').$$

The angle Θ must, in order to prevent slipping of the wall on its bed, always be less than the angle of friction or repose, φ^1, of wall and foundation or bed joints. If the wall is computed to be stable against rotation, this condition will generally be satisfied, except in the case of a leaning wall. If it is found to be unstable in regard to slipping, the wall must be made heavier than is required for stability against rotation; or the base of the wall must be made to pitch back at such an angle γ that $\Theta - \gamma$ must be less than φ^1 in the required ratio. In most cases of Quay-walls this device would necessitate the use of coffer-dams or diving-bells; in cases of walls laid by divers on pile work or random stone, it could not be used with economy.

If we rely upon this device, great care is required in making the foundations, for they must be perfectly stable against the tendency to push them out horizontally,—especially in the case of a leaning wall,—which is greater as the wall is lighter. When piles are used, they should be driven perpendicularly to the plane of slope, i. e., at an angle γ with the vertical.

IV. Factors of Stability; Coefficient $m = \frac{1}{2} - q^1$.

1°. In respect to rotation:

It is evident that the wall should, if possible, be so designed that there shall at least be no tendency, at any point of the bed-joint, to open; *i. e.*, that the back edge of the wall should never be entirely relieved of pressure.

If the pressure be taken as an uniformly varying struss, we can determine the value of m and q^1, as follows:

The base being rectangular and the resultant pressure upon it P, cutting it one side of its centre, the total effect upon the base will be equivalent to an uniformly distributed force P, and a moment $P\,q^1\,b$, where $q^1\,b$=deviation of the said force from the centre.

Let us consider an unit of length of wall:

The area of the base is then b, and the pressure per unit of area of base is $\frac{P}{b}$. The pressure per unit of area of base, due to the moment $P\,q^1\,b$, must evidently not be greater than $\frac{P}{b}$; for having an opposite sign on the back edge of the wall, if it is just equal to it, the pressure at that point will be entirely relieved. Now the stress due to this moment at the extremities or edges of the base is:

$$S=\frac{P\,q^1\,b\,y}{I};$$

Where I=moment of inertia of section of base$=\frac{b^3}{12}$;

y=distance of point of most intense pressure from centre of gravity of the section of base$=\frac{1}{2}\,b$:

$$\frac{P\,q^1\,b\,y}{I}=\frac{6\,P\,q^1}{b}=\frac{P}{b};$$

$$\therefore\ q^1=\tfrac{1}{6}\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(19)}$$

$$m=\tfrac{1}{3}\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(20)}.$$

The English engineers generally adopt the following values:

$$q^1=.375=\tfrac{3}{8};\ \text{or}\ m=\tfrac{1}{8};$$

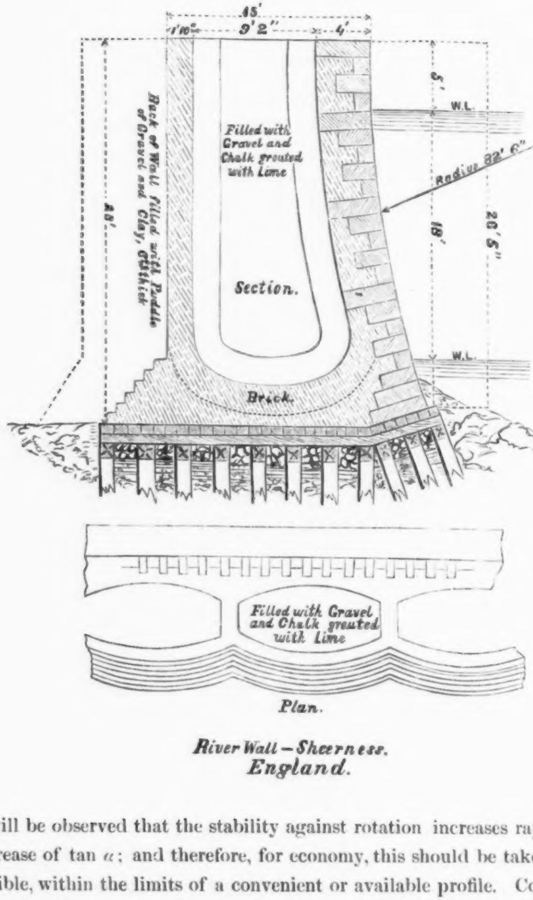
the French engineers generally adopt the following values:

$q^1=\frac{3}{16}$ to $\frac{1}{4}$; or $m=\frac{1}{5}$ to $\frac{1}{4}$. (See Rankine's Civ. Eng.)

For treacherous foundations or soft bottoms, or material, the value of q^1 should not be greater than, nor that of m less than, the theoretical value. In some cases even greater stability is requisite, in order to bring the line of resultant pressure on the base nearly vertical. In cases of firm and reliable foundations the values usually adopted by English engineers may be used, though those of the French engineers are more nearly theoretical, and their walls heavier.

A remarkable example of the necessity of adopting in certain cases a very large value for m is furnished in the Sheerness (England) wall designed by Rennie The bottom is of the most treacherous character, and the foundation is pile-work requiring a very large value of m to insure stability. The wall was therefore built hollow and afterwards filled with puddle, in order to obtain the desired stability most economically. A sketch of this wall is annexed.

Fig. 2.



River Wall — Sheerness.
England.

It will be observed that the stability against rotation increases rapidly wi the increase of tan α; and therefore, for economy, this should be taken as lar as possible, within the limits of a convenient or available profile. Considerab economy is gained by adopting offsets in the profile of the back; as the weig of the overlying earth takes the place of a proportional weight of wall. Ca must, however, be taken that the offsets be not so large as to render the upp portion of the wall less stable than the lower. The chief difficulty, however, soft or treacherous foundations, arises at the lowest level of the wall; *i. e.*, the foundation; and the stability at the upper levels will generally be great than that at the lower levels. In some difficult cases, where the base requi

great breadth, several altar courses are introduced, so as to form a *toe* at the outer edge, and to so increase the value of m.

But there is another limit which must be satisfied, and which is generally, in ordinarily favorable cases, satisfied by the above;—which is, that the pressure on no part of the base shall exceed the safe pressure of the material, either of the foundation or wall.

For this we cannot do better than to quote Professor Rankine (Civ. Engineering, p. 397, eq. 1), Taking

R= resultant force;

b= breadth of base in feet;

t= thickness of base = unity in our case;

f^1= the greatest safe pressure in lbs. to the square foot, (being about $\frac{1}{8}$ of the crushing force);

Then,

$$f^1=2\,R\div(\tfrac{3}{2}-3\,q^1)\,b\,t\text{; and therefore}$$

$$q^1=\tfrac{1}{2}-\frac{2\,R}{3\,f^1\,b\,t};$$

or:

$$q^1=\tfrac{1}{2}-\frac{2\,R}{3\,f^1\,b}\qquad\qquad(21);$$

and:

$$m=\tfrac{1}{2}-q^1=\frac{2\,R}{3\,f^1\,b}\qquad\qquad(22).$$

R is to be found by the method given above. The wall where there is a suspicion that this limit is reached, must be tested by formulas (21) or (22).

In case the base is inclined, the portion of the wall below the horizontal $H\,D$ is to be considered as part of the foundations;—the formulæ applying only to the portion above the said line. The inclination of the base being given to secure the wall against slipping, it is not necessary to consider the portion of it below $H\,D$ in computing for stability against rotation.

The tendency of the earth pressure to open the bed-joints at the back slope is greater on the level $H\,D$ than below it, in such cases. By introducing γ into the formulæ, they can be readily modified so as to include the portion of the wall below $H\,D$, becoming however more complicated, and yet *not* more nearly representative of the practical question.

2°. In respect to slipping:

The stability against slipping depends mainly upon the weight of the wall and overlying earth. As has been remarked, tan Θ, or tan $(\Theta-\gamma)$, must always be less than tan φ^1 (tangent of angle of friction of wall on the foundation or bed); *i. e.*, the obliquity of the resultant R to the vertical, Θ, or $\Theta-\gamma$, must

always be less than the angle of repose or friction φ^1 of the wall on the foundation or bed considered.

To guard against slipping, tan θ, or tan $(\theta-\gamma)$, should be small enough to place the stability beyond doubt.

Poncelet and others have assumed tan $\varphi^1=\frac{7}{10}=$ tan $35°$. Tan φ^1 is generally taken for *ordinary cases* from $\frac{65}{100}$ to $\frac{60}{100}$; or φ^1 from $33°$ to $31°$. In cases of ordinary retaining walls (not immersed) the value of tan θ cannot with economy be reduced below tan $\theta=\frac{1}{2}$,—unless with very broad offsets or a steep batter at the back, and the consequent great addition of weight of the overlying earth; necessitating also a very broad foundation. With deeply immersed walls with water-tight joints and foundations, however, the pressure of water on the wall reduces the tendency to slip, and consequently tan θ is likewise reduced, (see Ex. 1.) In ordinary cases, therefore, the limits may be assumed to be: tan θ not $> \frac{7}{10}$ nor $< \frac{5}{10}$.

But in cases of treacherous foundations liable to push out horizontally, as pile-work in soft soils, θ should be fixed so as to bring the magnitude of the horizontal component of R sufficiently within the limits of safety in that regard. The Sheerness wall is a case in point. In some cases of Quay-walls the requirements of the stability against slipping must rule the problem. The following tables give values of φ and tan φ for different materials.

[*Extract from "Rankine's Civil Engineering."*]

SURFACES.	φ	$f=\tan\varphi$	$\frac{1}{f}$
Dry masonry and brickwork	31° to 35°	0.6 to 0.7	1.67 to 1.4[illegible]
Masonry and brickwork with wet mortar	25½°	0.47	2.1
Masonry and brickwork with slightly damp mortar	36½°	0.74	1.35
Wood on stone	22°	about 0.4	2.5
Iron on stone	35° to 16⅔°	0.7 to 0.3	1.43 to 3.3[illegible]
Masonry on dry clay	27°	0.51	1.96
" " moist clay	18¼°	0.33	3
Earth on earth	14° to 45°	0.25 to 1.0	4 to 1
" " " dry sand; Clay and mixed earth	21° to 37°	0.38 to 0.75	2.63 to 1.3[illegible]
Damp clay	45°	1.0	1
Wet clay	14° to 17°	0.25 to 0.31	3.23 to 4
Shingle and gravel	35° to 48°	0.7 to 1.11	1.43 to 0.[illegible]

[*Extract from "Thomas Stevenson on Harbors.*"]

'Mr. George Rennie found that $\frac{8}{10}$ths of the weight were required to drag a block of stone over the roughly dressed floor of a quarry, and that the "voussoirs" of the London Bridge began to glide over each other at a slope of from 33° to 34°.

"I made a few experiments on the friction of small, polished, aislar blocks of freestone, and the mean gave $\frac{5}{10}$ths of the whole weight moved as the coefficient of friction of such stones over a similarly polished freestone block. The *power* required to *extract* a polished freestone block out of its place in a column consisting of different numbers of blocks was also tried. The coefficient of friction in extracting any of the blocks from the column was 1.083 or about *twice* the amount required for moving the whole mass (including the stone extracted). As the blocks in a pier are often surrounded by water, it appeared desirable to ascertain the friction in water as well as in air, and the following numbers show the results, as also the coefficients for stones in different styles of dressing.

Kinds of Dressing and of Materials.	Angles at which blocks begin to move.		Tangents of angles of friction in air, or coefficients of friction f.
	In water φ^1	In air φ^1	
Polished freestone on polished freestone.........	28	28	.53
Longitudinal axing on cross-broaching*....	....	$37\frac{1}{2}$	.77
Cross-axing on cross-broaching....	....	$38\frac{1}{4}$	.79
Closely axed greenstone on scabbled freestone....	39	35	.81
Longitudinal broaching on pick-dressing.........	....	$39\frac{1}{4}$	.82
Closely axed greenstone on scabbled freestone....	....	40	.84
Cross close-axing on pick-dressing..............	$41\frac{1}{3}$	$40\frac{1}{8}$	.84
Longitudinal broaching on longitudinal broaching	41	$41\frac{1}{3}$	.88
Closely pick-dressed freestone on closely pick-dressed freestone......................	51	43	.94

"As the blocks which were used for these experiments were of too small sizes to give anything more than a rough comparative valuation for air and water, I got stones of about a ton in weight, dressed in different styles of workmanship, at the sight of Mr. Robert Kinnear, inspector of works, when the following results were obtained :

* "Broaching is a style of work peculiar to Scotland, and consists of a number of narrow parallel ridges running close to each other. They are made with a small pointed chisel, and extend over the whole of the face work between the drafts."

	Angles at which the stones began to move.	Coefficient of friction.
Droving on droving waves parallel to line of slip...	33° 8′	.65
Broaching on broaching with stripes or ridges running parallel to line of slip..............	34° 3′	.68
Droving on droving waves across to line of slip....	35° 57′	.73
Broaching on broaching, with stripes or ridges lying crossways to the direction of the slip....	36° 6′	.73
Rough pick-dressing on rough pick-dressing.......	38° 11′	.79

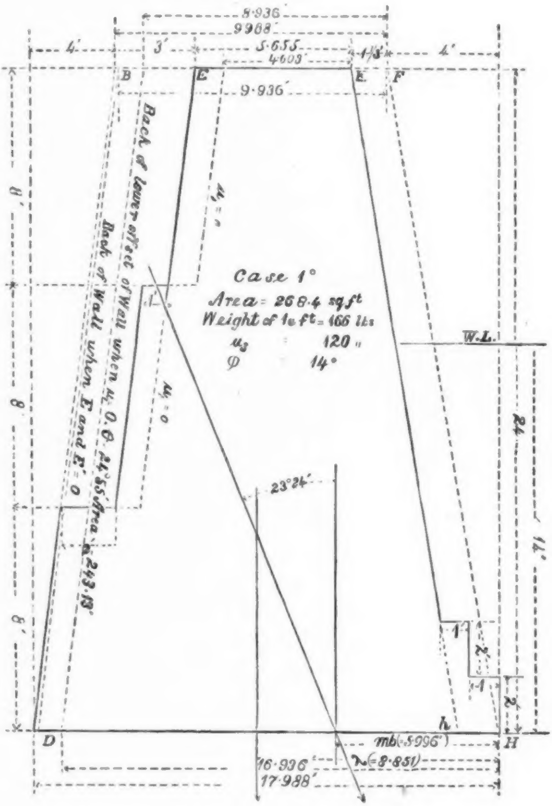
TABLE OF VALUES OF φ^1, DETERMINED BY THE WRITER AT THE YARD OF THE DEPARTMENT OF DOCKS, N. Y. CITY.

Materials.		φ^1 for Quiescence dry.	φ^1 for Motion dry.	φ^1 for Quiescence wet.	φ^1 for Motion wet.
Granite on Granite:					
1°. Good rough pointed ashlar, surface nowhere more than $\frac{3}{8}''$ out of plane	Block 2′ × 2′ × 4′.	35°	24½°		
2°. "8 cut," or fine hammered, surface:	Small Blocks.				
a. Grooves of hammer horizontal		34½°	25°		
b. " " " crossed.........		34°	30°		
c. " " " in line of motion.		34°	30°		
3°. "10 cut" work, or very finely dressed:					
a. Grooves of hammer horizontal.......		33°	32°	33¼°	29
b. " " " crossed.........		31¼°	30½°		
c. " " " in line of motion		31½°	30½°		
Black Marble on Granite:					
a. Granite, "10 cut," marble polished..		30½°	28°		
b. " " " hammered to about "8 cut" work........		35°	28°		
Granite on smooth faced or pressed Béton Coignét:					
a. Granite, "8 cut,"................		30½°	27¼°		
b. " rough pointed............		30°	27¼°		
Béton Coignét on Béton Coignét:					
Smooth pressed		33¼°	31½°		

The water in the 3d experiment was kept constantly running on and between the surfaces. It will be observed that the friction, when wet, is about the same as when dry.

EXAMPLES.

Case 1.



a. Let $\tan \alpha = \tan \beta = \frac{1}{6} =$ batter;

$m = \frac{1}{3}$;

$u_2 = 166$ lbs.; $u_1 = 64.1$ lbs.; $u_3 = 120$ lbs.;

$\varphi = 14°$; $h = 24$ feet; $d = 14$ feet.

Let there be two offsets at the back, placed at points dividing the height, h, into equal divisions; make the upper one 1 foot wide, and the lower one 2 feet wide.

Let there be two steps at the toe of the wall, each 2 feet high and 1 foot broad, as in figure. Required the value of a^1.

Let b_1, b_2, b_3, &c., be the widths of the offsets in the back, numbering from the top one; and n=number of divisions of height, h, made by offsets, or number of offsets plus 1. Then:

$$c=\frac{b_1\left(\frac{b_1}{2}+b_2+b_3+\&c.+\frac{2n-1}{2n}h\tan\beta\right)+2b_2\left(\frac{b_2}{2}+b_3+\&c.+\frac{2n-2}{2n}h\tan\beta\right)+\&c.}{b_1+2b_2+\&c.}$$

$$g=b_1+b_2;\quad k=1.667=Hh=FK.$$

In this case

$$c=\frac{1\times(\frac{1}{2}+2+\frac{5}{6}\times4)+2\times2\ (1+\frac{2}{3}\times4)}{1+2\times2}$$

$$=4.1 \text{ feet};$$

$$E=40\ (u_2-u_3)=40\ (166-120)=1840 \text{ lbs.};$$

and

$$c\,E=7544.$$

$$E^1=27.33\times166=4537 \text{ lbs.};$$

$$c^1=3 \text{ feet};$$

and

$$c^1E^1=3\times4537=13611.$$

If $\tan\beta=\frac{1}{6}=\tan 9°\ 27'\ 50''$, and $\varphi=14°$;

$$u_4=\frac{(\cos\beta-\sin\varphi)^2}{\cos^2(\beta+\varphi)}\cdot u_3=\frac{(.98639-.24192)^2}{.91731^2}\times120=79 \text{ lbs.}$$

If $\tan\beta=\frac{1}{8}=\tan 7°\ 7'\ 15''$;

$$u^4=\frac{(.99230-.24192)^2}{.92385^2}\times120=78 \text{ lbs.}$$

If $\tan\beta=0$:

$$u_4=\frac{1-\sin\varphi}{1+\sin\varphi}\cdot u_3=\frac{1-.24192}{1+.24192}\times120=73 \text{ lbs.}$$

If $\tan\beta=-\frac{1}{6}$:

$$u_4=\frac{(\cos\beta-\sin\varphi)^2}{\cos^2(\beta-\varphi)}\cdot u_3=\frac{(.98639-.24192)^2}{.99546^2}\times120=67 \text{ lbs.}$$

If $\tan \beta = -\frac{1}{8}$:

$$u_4 = \frac{(.99230 - .24192)^2}{.99279^2} \times 120 = 69 \text{ lbs.}$$

1°. First term of Second Member $= A$, eq. 11:

$$h^2 \tan \alpha \,[3\, u_2\, (1-2\, m) + u_4\, (1-m)] = 24^2 \times \tfrac{1}{6}\, [3 \times 166\, (1-\tfrac{2}{3}) + 79\, (1-\tfrac{1}{3})] \qquad = \quad 20992.032.$$

$$-u_1\, d \tan \alpha \,[4\, (1-2\, m)\, h + m\, d] = -\frac{64.1 \times 14}{6} (\tfrac{4}{3} \times 24 + \tfrac{1}{3} \times 14) \qquad = \quad -5484.139.$$

$$-2\, (1-m)\, E \qquad = \qquad -2\, (1-\tfrac{1}{3}) \times 1840 \qquad = \quad -2453.333.$$

$$+2\, m\, E^1 \qquad = \qquad \tfrac{2}{3} \times 4537 \qquad = \quad +3024.66.$$

$$2\left(u_2 h\, (1-2\, m) - u_1 (1-2\, m)\, d\,\right] = 2\left(166 \times \tfrac{24}{3} - \frac{64.1 \times 14}{3}\right] \qquad = \quad 2057.733.$$

$$\therefore\ A = -7.814.$$

2°. Quantities under the radical sign:

$$h^3 [u_4 \left(1 - (5 - 6\, m) \tan^2 \alpha\right) + 6\, u_2 \tan^2 \alpha\, (2\, m - 1)] =$$

$$\overline{24}^3 [79 \left(1 - (5 - \tfrac{6}{3}) \times \tfrac{1}{36}\right) + \frac{6 \times 166}{36} (\tfrac{2}{3} - 1)] \qquad = \quad 873593.854.$$

$$12\, (1-2\, m)\, d\, h^2 u_1 \tan^2 \alpha = 12\, (1-\tfrac{2}{3}) \times 14 \times \overline{24}^2 \times 64.1 \times \tfrac{1}{36} \qquad = \quad 57433.600.$$

$$6\, h\, [2\, (1-m) E \tan \alpha - 2\, m \tan \alpha\, E^1 + m\, u_1 d^2 \tan^2 \alpha] =$$

$$6 \times 24 \left[\tfrac{4}{3} \times \tfrac{1840}{6} - \tfrac{2}{3} \times \tfrac{4537}{6} + \tfrac{1}{3} \times \frac{64.1 \times 14^2}{36}\right] \qquad = \quad 3039.84.$$

$$-u_1 d^3 (1 + \tan^2 \alpha) \qquad = \qquad -64.1 \times 14^3 (1 + \tfrac{1}{36}) \qquad = -180776.244.$$

$$-6\, c\, E \qquad = \qquad -6 \times 7544 \qquad = \quad -45264.$$

$$+6\, c^1 E^1 \qquad = \qquad +6 \times 13611 \qquad = \quad +81666.$$

$$3\, (1-2\, m)\, (u_2 h - u_1 d) \qquad = \qquad \qquad = \quad 3086.598.$$

$$A^2 = 61.059$$

$$\therefore\ a = -7.814 + \sqrt{255.8454 + 61.059},$$

$$= -7.814 + 17.802 = 9.988 \text{ feet.}$$

$$\therefore\ a^1 = a - (g + k) = 9.988 - 4.333,$$

$$= 5.655 \text{ feet.}$$

$$b=17.988 \text{ feet.}$$

Proof: Taking the moments about 0, we have;—

Moment of wall and overlying earth:

$$\underline{55728}\times 2.998-\underline{1840}\ (11.992-4.1)+\underline{4537}\ (5.996-3)=166141;$$

Moment of water pressure on face:

$$6368\left(4.731-\frac{1.971}{2}\right)=23855;$$

Moment of water pressure on base:

$$-16142\times 2.998=-48394;$$

Moment of earth pressure on back:

$$=-23061\ (8.11-1.971)=-141571;$$

$$+\text{moments}=189996;$$

$$-\text{moments}=189965;$$

$$\text{Error}=+\frac{31}{189965}.$$

To find θ and R:

$$Y=\Sigma\ P\sin\alpha=P_1\sin\alpha+P_2-P_3+P_4\sin\alpha=$$
$$=6368\times .164+55728-1840-4537-16142+23061\times .1644$$
$$=38047 \text{ lbs.}$$

$$X=\Sigma\ P\cos\alpha=P_1\cos\alpha-P_4\cos\alpha=$$
$$=(6368-23061)\times .986=-16459.$$

$$\therefore\ \tan\theta=-\frac{16459}{38047}=-.4327=\tan\ (-23^\circ\ 24'),$$

(to be laid off *above* 0 from vertical towards the back) which is safely within the limiting value of $\tan\varphi^1$.

$$R=\frac{38047}{.937}=40.600 \text{ lbs.}$$

Distance of centre of gravity of wall from H:

$$\lambda=\frac{b}{3}+\frac{\text{moment of wall about } 0}{\text{weight of wall}}$$
$$=\frac{17.988}{3}+\frac{127173}{44531}=5.996+2.855$$
$$=8.851 \text{ feet.}$$

Area of section of wall $=268\frac{4}{10}$ sq. feet.

b. If E and E^1 are made each $=0$, the section will be a plain trapezoid, and:

$$a=a^1=9.936 \text{ feet};$$

and the sectional area $=334\tfrac{46}{100}$ sq. feet.

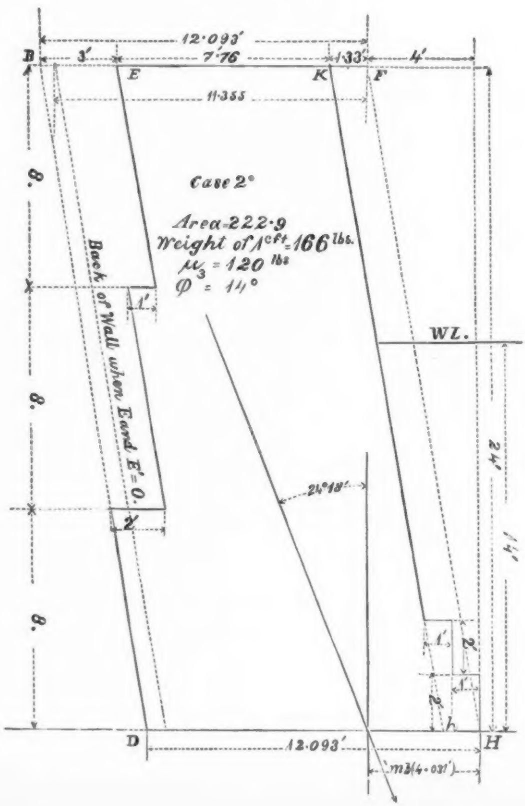
The comparison of these results shows how important the elements E and E^1 are in securing economy of material.

c. If $u_1=0$, *with* offsets, E and E^1:

$$a=9.936\,; \qquad a^1=4.603\,; \qquad b=16.936\,;$$

$$\text{sectional area}=243.\tfrac{13}{100} \text{ sq. feet}\,; \qquad \Theta=24^\circ\ 55'.$$

Case 2.



Let $\tan\beta=\tan(-\alpha)=-\frac{1}{6}$; *i. e.*, the back will slope back at an angle whose tangent $=\tan(-\alpha)=-\frac{1}{6}$. Let all other elements not dependent on α or β remain as in Case 1. In this case, then,

$$u_4=67 \text{ lbs. (see p. 210); and}$$

$$c=\frac{1\left(\frac{1}{2}+2-\frac{5}{6}\times 4\right)+2\times 2\left(1-\frac{2}{3}\times 4\right)}{1+2\times 2}=-1'.5.$$

1°. First term of Second Member $=A$, eq. 12:

$h^2\tan\alpha\,[u_2-u_4(1-m)]$	$=$	$24^2\times\frac{1}{6}\,[166-67\times\frac{1}{3}]$	$=$	11647.968
$-u_1\,m\,d^2\tan\alpha$	$=$	$-64.1\times\frac{1}{3}\times 14^2\times\frac{1}{6}$	$=$	-697.978
$-2(1-m)E$	$=$	$-2\times\frac{2}{3}\times 1840$	$=$	-2453.333
$+2\,m\,E^1$	$=$	$+\frac{2}{3}\times 4537$	$=$	3024.66
$2(1-2\,m)(u_2 h-u_1 d)$	$=$	$2\left(166\times\frac{24}{3}-64.1\times\frac{14}{3}\right)$	$=$	2057.66

$$\therefore\ A=-5.5994 \text{ feet.}$$

2°. Quantities under radical sign:

$(1+\tan^2\alpha)(h^3 u_4-u_1 d^3)$	$=$	$\frac{37}{36}\,(24^3\times 67-64.1\times 14^3)$	$=$	771159.755
$-6\,c\,E$	$=$	$-6\times -1.5\times 1840$	$=$	$+16560$
$+6\,c^1 E^1$	$=$	$+6\times 13611$	$=$	81666
$3(1-2\,m)(u_2 h-u_1 d)$	$=$		$=$	3086.598
A^2	$=$		$=$	31.349

$$\therefore\ a=-5.599+\sqrt{281.665+31.349}$$

$$=-5.599+17.692=12.093 \text{ feet.}$$

$$\therefore\ a^1=-12.093-4.333=7.760 \text{ feet.}$$

$$b=\ 12.093 \text{ feet.}$$

Proof:

Moment of wall and earth over offsets:

$$48179\times 4.015-1840\,(8.06+1.5)+4537\times(4.03-3)=+180521;$$

Moment of water pressure on face:

$$=6368\,(4.731-.663)=+25905;$$

Moment of water pressure on base:

$$=-10853\times 2.015=-21869;$$

Moment of water pressure on back:

$$=19562\ (8.11+1\ 325)=-184577\ ;$$

$$\left.\begin{array}{l}+\text{moments}=206426\ ;\\ -\text{moments}=206446\ ;\end{array}\right\}\qquad \text{error}=-\frac{20}{206446}.$$

To find θ and R:

$$Y=6368\times.1644+48179-1840-4537-19562\times.1644-10823$$
$$=28810\ \text{lbs.}$$

$$X=(6368-19562)\times.986=-13009\ \text{lbs.}$$

$$\therefore\ \tan\theta=\frac{X}{Y}=-.4515=\tan\ (-24^\circ\ 18').$$

If the base be inclined so that it shall be perpendicular to the face:

$$\theta-\gamma=\theta-\alpha=24^\circ\ 18'-9^\circ\ 28'=14^\circ\ 50'.$$

The wall is, however, safe against slipping, without inclining the base.

Sectional area $=222\tfrac{9}{10}$ sq. feet, showing considerable economy over Case 1.

If E and E^1 be made each $=0$, the section of the wall will be a simple parallelogram, and

$$a=a^1=11.355\ \text{feet}\ ;$$

$$\text{sectional area}=272\tfrac{5}{10}\ \text{sq. feet.}$$

Case 3.

(See Illustration on next page.)

Let $\beta=0$; and all the terms not dependent on β the same as in Case 1.

First term of Second Member—eq. 13:

$$h^2 u_2\,(2-3\ m)\tan\alpha \quad = \quad 24^2\times 166\times(2-1)\times\tfrac{1}{6} \quad = \quad 15936.$$

$$-u_1 d\left(2\ (1-2\ m)\,h+md\right)\tan\alpha=-64.1\times 14\times(\tfrac{2}{3}\times 24+\tfrac{1}{3}\times 14)\times\tfrac{1}{6}\ =-3091.094.$$

$$-2\ (1-m)\ E \quad = \quad -2\ (1-\tfrac{1}{3})\times 1840 \quad =-2453.333.$$

$$+2\ m\ E^1 \quad = \quad +2\times\tfrac{1}{3}\times 4537 \quad = \quad 3024.66.$$

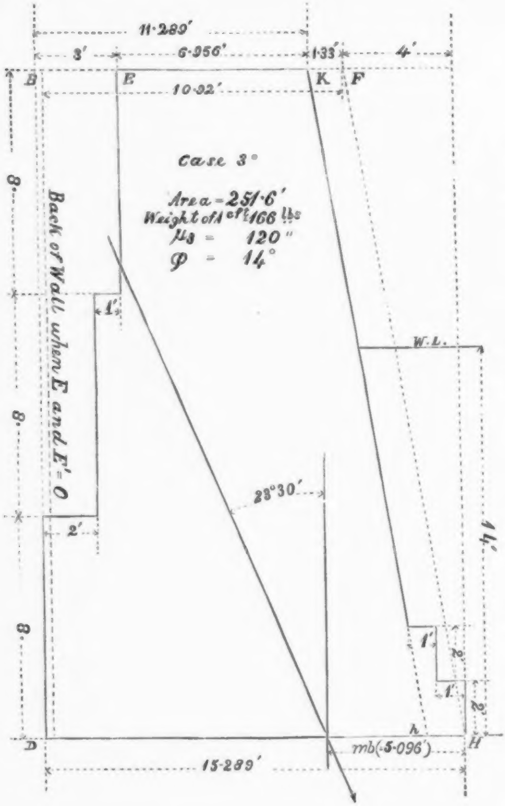
$$\div 2\ (1-2\ m)\ (u_2\,h-u_1\,d\,)= \quad = \quad \div 2057.66.$$

$$A=-6.52.$$

Quantities under the radical sign:

$$h^3\,[u_4+(3\ m-2)\ u_2\tan^2\alpha]=24^3\ [73+(1-2)\ 166\times\tfrac{1}{36}] \quad = \quad 945409.536.$$

$$3(1-2\ m)\ u_1\ d\ h^2\tan^2\alpha \quad =3\ (1-\tfrac{2}{3})\times 64.1\times 14\times 24^2\times\tfrac{1}{36} \quad = \quad 14358.4.$$



$$h\,[6\,(1-m)\,E\tan\alpha-6\,m\,E^1\tan\alpha+3\,m\,u_1\,d^2\tan^2\alpha];=$$

$$24\left[6\times\tfrac{2}{3}\times1840\times\tfrac{1}{6}-\tfrac{6}{3}\times4537\times\tfrac{1}{6}+\frac{64.1\times14^2}{36}\right]\;=\quad 1519.72$$

$$-u_1\,d^3\,(1+\tan^2\alpha)\quad=\quad-64\;1\times14^3\,(1+\tfrac{1}{36})\quad=-180776.24$$

$$-6\,c\,E+6\,c^1\,E^1\quad=\quad-6\times1.3\times1840+6\times13611\quad=\quad6731$$

$$3\,(1-2\,m)\,(u_2h-u_1d)=\quad\quad=\quad\div3086.58$$

$$A^2=42.51.$$

$$\therefore\quad a=-6.52+\sqrt{274.67+42.51}$$

$$=-6.52+17.809=11.289'.$$

$\therefore\ a^1 = a-(g+k) = 11.289-4.33 = 6.956$ feet; and

$b = 15.289$ feet.

Proof:

Moment of wall and earth:

$= 52944 \times 3.498 - 1840\ (10.192-1.300) + 4537\ (5.096-3) = 178340.6$;

Moment of water pressure on face:

$$=6368\ (4\ 731-0.838) = 24790.624;$$

Moment of water pressure on base:

$$=-13720 \times 2.548 = -34959;$$

Moment of earth pressure on back:

$$=-21024 \times 8 = -168192;$$

$$\left.\begin{array}{l} +\text{moments} = 203130; \\ -\text{moments} = 203151; \end{array}\right\} \qquad \text{error} = -\frac{21}{203130}; \qquad \text{mean error} = -\frac{10\frac{1}{2}}{203130}.$$

$X = P_1 \cos \alpha - P_4 \cos \alpha = +(6368 \times .986 - 21024) = -14745$ lbs.

$Y = P_1 \sin \alpha + P_2 - P_3 + P_4 \sin \alpha = 6368 \times .1644 + 52944 - 1840 - 4537 - 13720$
$= 33794.$

$$\tan\ \Theta = \frac{X}{Y} = -\frac{14745}{33894} = -.4350 = \tan\ (-23^\circ\ 30').$$

Sectional area $= 251\tfrac{6}{16}$ sq. feet.

If E and E^1 each equal 0:

$a = a^1 = 10.920$ feet; and $b = 14.920$ feet;

sectional area $= 310\tfrac{8}{16}$ sq. feet.

Case 4.

(See Illustration on next page.)

Let the section be rectangular, $\tan \alpha = \tan \beta = 0$; the other elements not dependent on α or β remaining as before. Then,

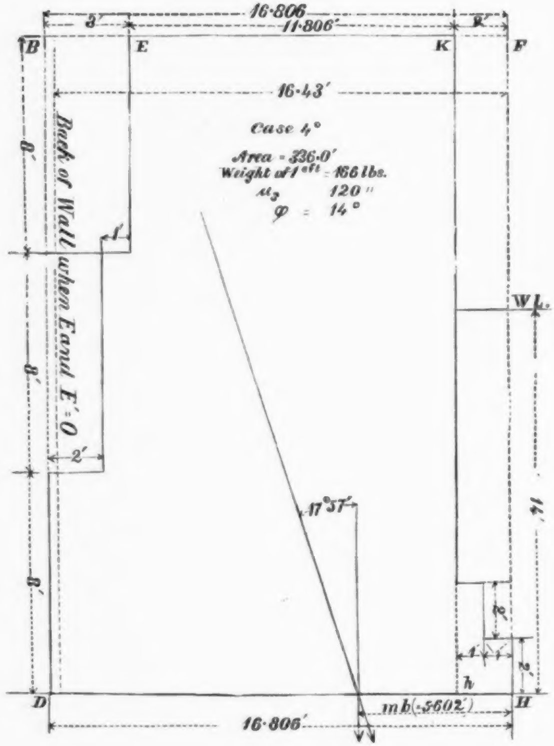
$$u_4 = 73 \text{ lbs.}$$

$$e = \frac{1\ (\frac{1}{2}+2) + 2 \times 2 \times (1)}{1 + 2 \times 2} = +1.3 \text{ feet.}$$

First term of Second Member $= A$, eq. 14:

$(1-m)\ E - 2\ m\ E^1$	$=$	$2 \times \frac{2}{3} \times 1840 - \frac{2}{3} \times 4537$	$=$	$-571.33.$
$(1-2\ m)\ (u_2 h - u_1 d)$	$=$	as before	$=$	$2057.66.$

$$A = -.277.$$



$$\ell^3 u_4 - u_1 d^3 \quad = \quad 24^3 \times 73 - 64.1 \times 14^3 \quad = \quad 833261$$

$$-6\,c\,E \quad = \quad -6 \times 1.3 \times 1840 \quad = \quad -143$$

$$-6\,c^1\,E^1 \quad = \quad +6 \times 13611 \quad = \quad +816$$

$$\ell^2 \quad = \quad \quad = \quad .07$$

$$(1-2\,m)\,(u_2 h - u_1 d) = \quad \quad = \quad 3086.5$$

$$\therefore \quad a = 17.083 - 0.277 = 16.806 \text{ feet.}$$

$$a^1 = 16\ 806 - 5 \qquad = 11.806 \text{ feet.}$$

Proof:

Moment of wall and earth :

$$66955.1 \times 2.801 - 1840\,(11.203 - 1.3) + 4537\,(5.601 - 3) = +181120;$$

Moment of water pressure on face:

$$6287 \times 4.667 = +29341;$$

Moment of water pressure on base:

$$-15082 \times 2.801 = -42245;$$

Moment of earth pressure on back:

$$-21024 \times (8.00) = -168192;$$

$$\left.\begin{array}{l} +\text{moments} = 210461; \\ -\text{moments} = 210437; \end{array}\right\} \qquad \text{error} = +\frac{24}{210437}.$$

To find θ and R:

$$X = (6287 - 21024) = -14737 \text{ lbs.}$$

$$Y = 66955 - 1840 - 4537 - 15082 = 45496.$$

$$\therefore \tan \theta = -\frac{14737}{45496} = -.3239 = \tan (17^\circ\ 57').$$

Sectional area = 336 sq. feet.

If E and E^1 are each $=0$:

$$a = a^1 = 16.43 \text{ feet};$$

$$\text{sectional area} = 394\tfrac{3}{10} \text{ sq. feet.}$$

It will be remarked that all of the above walls are designed to stand safely under the worst conditions. These conditions are such, however, as are very likely to obtain in Quay-walls, exposed as they are to the effects of water, continual vibrations, &c.

The following table exhibits the relative economy of the above sections:

CASES.	Section: with E and E^1.	θ	Section: E and $E^1=0$.
Case 2.	$222\frac{9}{10}$	24° 18′	$272\frac{5}{10}$.
Case 3.	$251\frac{6}{10}$	23° 30′	$310\frac{8}{10}$.
Case 1.	$268\frac{4}{10}$	23° 24′	$334\frac{46}{100}$.
Case 4.	336	17° 57′	$394\frac{3}{10}$.

COUNTER-FORTS.

The preceding formulæ are applicable to the case of counter-fort walls. this case the weight of the counter-fort, reduced to equivalent weight per fo lineal of length of wall, is to be included in the terms E and $c\,E$; and it evident it will have the effect of reducing these terms.* To obtain the equiv lent weight of a counter-fort per foot lineal, it is only necessary to multiply th weight of one counter-fort by the ratio of its thickness to the distance apart counter-forts from centre to centre. Counter-forts are of very doubtful efficienc In cases of treacherous foundations they certainly should not be used, account of the danger of unequal settlement. They evidently do not increas but diminish the stability against slipping; and their effect is but small favor of stability against rotation.

We can estimate the saving of material due to their use, as follows:

Let M=moment of wall about heeling point H.

M_x=moment of an equivalent wall with counter-forts, exclusive moment of counter-fort, about same point.

M_c=moment of counter-forts about same point,

$$\therefore\ M=M_x+M_c \ldots\ldots\ldots\ldots\ldots\ldots (2$$

Now for a trapezoidal section; making $u_2=1$,

$$M=\frac{h}{6}\left[3\,a^1\left(a^1+h\,(2\tan\alpha+\tan\beta)+h^2\,(2\tan\alpha+\tan\beta)\,(\tan\alpha+\tan\beta)\right]\right.$$

$$=\frac{h}{6}\left[3\,a^1\,(a^1+h\,p)+h^2\,p\,q\right] \ldots\ldots\ldots\ldots\ldots\ldots (24$$

Let b^1=mean thickness of counter-forts;

c=distance apart of counter-forts from centre to centre;

l=length of counter-forts or dimension perpendicular to face of wall

a=top width of counter-fort wall, exclusive of counter-forts;

and let the back be taken vertical, and counter-forts rectangular.

There then results:

$$M=M_x+M_c=\left(a+h\tan\alpha-\frac{\frac{1}{3}\,h^3\tan^2\alpha+a\,h^2\tan\alpha+a^2\,h}{2\,a\,h+h^2\tan\alpha}\right)\tfrac{1}{2}\,(2\,a\,h+h^2\tan\alpha)+$$

$$\left(a+h\tan\alpha+\frac{l}{2}\right)\frac{b^1}{c}\cdot l\,h \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

---

* Then will E = weight of material of wall lying between back profile of wall and vertic through back edge D—earth lying between same lines + *reduced weight of counter-fort;* and c = distance from D to centre of gravity of E.

Placing $\frac{b^1}{c}=r$, and reducing:

$$a=-(h\tan\alpha+rl)+\sqrt{\frac{2M}{h}-\tfrac{2}{3}h^2\tan^2\alpha-2rlh\tan\alpha-rl^2+(h\tan\alpha+rl)^2}$$

or placing s = batter of face for height h:

$$a=-(s+rl)+\sqrt{\frac{2M}{h}-\tfrac{2}{3}s^2-2rls-rs^2+(s+rl)^2}\;\ldots\ldots\ldots\ldots\ldots\ldots\;(26).$$

Example:

Let $a^1=4$ feet. $h\tan\alpha=h\tan\beta=4$ feet.

$\therefore\ b=12$ feet.

Let $b^1=4$ feet.

$l=(\frac{2}{10}\times h+2 \text{ feet})=6.8$ feet (by Vauban's rule).

$c=16$ feet.

Now M by (24) = 1152.

Introducing the above values in (26):

$$a=3\,926 \text{ feet.}$$

Weight of counter-fort wall per lineal foot =

$$\frac{3.926+7.926}{2}\times 24+\frac{6.8\times 24\times 4}{16}$$

$$=183.024.$$

$$\text{Saving}=\frac{24\times\frac{(4+12)}{2}-183.024}{24\times\frac{4+12}{2}}=.046$$

$4.\frac{6}{10}$ per cent.

The best method of cheapening the wall, when great stability is required, is evidently the one used by Rennie in the case of the Sheerness wall already referred to; *i. e.*, making the wall hollow, and filling it with cheap materials.

In the above estimate of the effects of counter-forts the aid from friction has been omitted, because it is very small and unreliable.

The distance to which the effect of a counter-fort is transmitted cannot with certainty be estimated; it depends mainly upon the sizes of the stones used and their bond.

By taking the proper values of u_4, h, and E, the formulæ can be adapted to the case of surcharged walls, when the top surface of the earth has a slope.

APPEND

$$a=-\frac{h^2\,[u_2\,(p-3\,m\,q)+u_4(1-m)\tan\beta]-2\,(1-2\,m)\,u_1 q\,d\,h-u_1\,m\,d^2\tan\alpha-2\,(1-m)\,E+2\,m\,E^1}{2\,(1-2\,m)\,(u_2\,h-u_1\,d)}+\sqrt{\frac{h^3\,[u_4\Big(1+\tan^2\beta-3\,(1-m)\,q\,\tan\beta\Big)+(3\,m\,q^2}{}}$$

$$a=-\frac{h^2[3\,u_2\tan\alpha\,(1-2\,m)+u_4(1-m)\tan\alpha]-u_1 d\tan\alpha\Big(4\,(1-2\,m)\,h+m\,d\Big)-2\,(1-m)\,E+2\,m\,E^1}{2\,(1-2\,m)\,(u_2\,h-u_1\,d)}+\sqrt{\frac{h^3[u_4\Big(1-(5-6\,m)\tan^2\alpha\Big)+6\,u_2\tan^2\alpha(2m-1}{}}$$

$$=-\frac{h^2\tan\alpha\,[3\,u_2\,(1-2\,m)+u_4\,(1-m)]-u_1\,d\tan\alpha\,\Big(4\,(1-2\,m)\,h+m\,d\Big)-2\,(1-m)\,E+2\,m\,E^1}{2\,(1-2\,m)\,(u_2\,h-u_1\,d)}+\sqrt{\frac{h^3[u_4\Big(1-(5-6\,m)\tan^2\alpha\Big)+6\,u_2\tan^2\alpha\,(2\,m-1}{}}$$

$$a=-\frac{h^2\tan\alpha\,[u_2-u_4\,(1-m)]-u_1\,m\,d^2\tan\alpha-2\,(1-m)\,E+2\,m\,E^1}{2\,(1-2\,m)\,(u_2\,h-u_1\,d)}+\sqrt{\frac{h^3 u_4(1+\tan^2\alpha)-u_1 d^3(1+\tan^2\alpha)-6\,c\,E+6\,c^1 E^1}{3\,(1-2\,m)\,(u_2\,h-u_1\,d)}+A^2}=-\frac{h^2\tan\alpha\,[u_2}{}$$

$$a=-\frac{h^2 u_2(2-3\,m)\tan\alpha-u_1 d\Big(2\,(1-2\,m)\,h+m\,d\Big)\tan\alpha-2\,(1-m)\,E+2\,m\,E^1}{2\,(1-2\,m)\,(u_2\,h-u_1\,d)}+\sqrt{\frac{h^3\,[u_4+(3\,m-2)\,u_2\tan^2\alpha]+3\,(1-2\,m)\,u_1\,d\,h^2\tan^2\alpha+h\,[6\,(1-}{3\,(1-}}$$

$$a=-\frac{h^2\,[u_2\,(p-3\,m\,q)+u_4\,(1-m)\tan\beta]-2\,(1-m)\,E+2\,m\,E^1}{2\,u_2\,h\,(1-2\,m)}+\sqrt{\frac{h^3\,[u_4\Big(1+\tan^2\beta-3\,(1-m)\,q\tan\beta\Big)+(3\,m\,q^2-p\,q)\,u_2]+6\,h\,[(1-m)q\,E-6}{3\,u_2\,h\,(1-2\,m)}}$$

$$a=-\frac{h^2\tan\alpha\,[3\,u_2\,(1-2\,m)+u_4\,(1-m)]-2\,(1-m)\,E+2\,m\,E^1}{2\,u_2\,h\,(1-2\,m)}+\sqrt{\frac{h^3\,[u_4\Big(1-(5-6\,m)\tan^2\alpha\Big)+6\,u_2\tan^2\alpha\,(2\,m-1)]+12\,h\,\;[(1-m)\,E\tan\alpha-}{3\,u_2\,h\,(1-2\,m)}}$$

$$a=-\frac{h\tan\alpha\,[3\,u_2\,(1-2\,m)+u_4\,(1-m)]}{2\,u_2\,(1-2\,m)}+\sqrt{\frac{h^3\,[u_4\Big(1-(5-6\,m)\tan^2\alpha\Big)+6\,u_2\tan^2\alpha\,(2\,m-1)]}{3\,u_2\,h\,(1-2\,m)}+A^2}\;\ldots\ldots\ldots\ldots\ldots\ldots(11b).\qquad a=-\frac{h^2\tan\alpha}{}$$

$$a=-\frac{h^2\,u_2\,(2-3\,m)\tan\alpha-2\,(1-m)\,E+2\,m\,E^1}{2\,u_2\,h\,(1-2\,m)}+\sqrt{\frac{h^3\,[u_4+(3\,m-2)\,u_2\tan^2\alpha]+6\,h\,[(1-m)E\tan\alpha-mE^1\tan\alpha]-6\,cE+6\,c^1E^1}{3\,u_2\,h\,(1-2\,m)}+A^2}\;\ldots\ldots\ldots$$

$$a=-\frac{h\,(2-3\,m)\tan\alpha}{2\,(1-2\,m)}+\sqrt{\frac{h^2\,\Big(u_4+(3\,m-2)\,u_2\tan^2\alpha\Big)}{3\,u_2\,(1-2\,m)}+A^2}=h\left(\sqrt{\frac{u_4+(3\,m-2)\,u_2\tan^2\alpha}{3\,u_2\,(1-2\,m)}+\left(\frac{A}{h}\right)^2}-\frac{(2-3\,m)\tan\alpha}{2\,(1-2\,m)}\right)\ldots\ldots\ldots$$

DIX.

$$\ldots + (3\,m\,q^2 - p\,q)\,u_2] + 3\,(1-2\,m)\,h^2 u_1 d\,q^2 + h\,[6\,(1-m)qE - 6\,m\,qE^1 + 3\,m\,q\,u_1 d^2 \tan\alpha] - u_1 d^3 (1+\tan^2\alpha) - 6\,c\,E + 6\,c^1 E^1 \over 3\,(1-2\,m)\,(u_2\,h - u_1\,d)} + A^2 \quad \ldots (10).$$

$$\left.\begin{array}{l} \ldots\alpha(2m-1)] + 12(1-2m)dh^2 u_1 \tan^2\alpha + h\,[12(1-m)E\tan\alpha - 12\,m\tan\alpha E^1 + 6\,m u_1 d^2 \tan^2\alpha] - u_1 d^3(1+\tan^2\alpha) - 6cE + 6c^1E^1 \over 3\,(1-2\,m)\,(u_2\,h - u_1\,d)} + A^2 \\ \ldots(2\,m-1)] + 12(1-2\,m)\,d\,h^2 u_1 \tan^2\alpha + 6\,h\,[2\,(1-m)E\tan\alpha - 2\,m\tan\alpha E^1 + m u_1 d^2 \tan^2\alpha] - u_1 d^3(1+\tan^2\alpha) - 6\,cE + 6\,c^1 E^1 \over 3\,(1-2\,m)\,(u_2\,h - u_1\,d)} + A^2 \end{array}\right\} \ldots (11).$$

$$\ldots\text{an}\,\alpha\,[u_2 - u_4\,(1-m)] - u_1\,m\,d^2\tan\alpha - 2\,(1-m)\,E + 2\,m\,E^1 \over 2\,(1-2\,m)\,(u_2\,h - u_1\,d)} + \sqrt{\frac{(1+\tan^2\alpha)\,(h^3\,u_4 - u_1\,d^3) - 6\,c\,E + 6\,c^1\,E^1}{3\,(1-2\,m)\,(u_2\,h - u_1\,d)} + A^2} \ldots (12);$$

$$\ldots [6\,(1-m)\,E\tan\alpha - 6\,m\,E^1\tan\alpha + 3\,m\,u_1\,d^2\tan^2\alpha] - u_1\,d^3\,(1+\tan^2\alpha) - 6\,c\,E + 6\,c^1\,E^1 \over 3\,(1-2\,m)\,(u_2\,h - u_1\,d)} + A^2 \ldots\ldots (13).$$

$$\ldots)q\,E - 6\,q\,mE^1] - 6\,c\,E + 6\,c^1 E^1 \ldots + A^2 \ldots\ldots (10a).$$

$$\ldots\tan\alpha - m\,E^1\tan\alpha] - 6\,c\,E + 6\,c^1\,E^1 \ldots + A^2 \ldots\ldots (11a).$$

$$\ldots h^2\tan\alpha\,[u_2 - u_4\,(1-m)] - 2\,(1-m)\,E + 2\,m\,E^1 \over 2\,u_2\,h\,(1-2\,m)} + \sqrt{\frac{(1+\tan^2\alpha)\,h^3\,u_4 - 6\,c\,E + 6\,c^1\,E^1}{3\,u_2\,h\,(1-2\,m)} + A^2} \ldots\ldots (12a).$$

$$\ldots\ldots (13a).$$

$$\ldots\ldots (13b).$$

LVIII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

TESTS OF BRIDGE IRONS.

A Paper by J. DUTTON STEELE, C. E., Member of the Society,

READ AT THE FIFTH ANNUAL CONVENTION, IN LOUISVILLE, KY., MAY 21ST AND 22D, 1873.

CHARLES HERMANY, C. E., IN THE CHAIR.

The first test was of chord links made of Phœnixville "Best Best," or double-refined new iron, 3 by 3 inches section, with upset eyes of the Phœnixville standard size, having a section round the eyes of 33 per cent. in excess of the section of the bar. The specifications required that the links should have an elastic strength of 24,000 lbs., and a breaking strength of 50,000 lbs. per square inch, with eyes strong enough to break the body of the bar. The testing machine was a lever with weights.

First application, 20,020 lbs. per square inch; stretched in 10 feet $\frac{1}{16}$-inch, and recovered.

Second application, 23,600 lbs. per square inch; stretched $\frac{3}{32}$-inch, and recovered.

Third application, 50,287 lbs. per square inch; stretched $\frac{5}{8}$-inch, and recovered $\frac{1}{8}$-inch.

Fourth application, 50,287 lbs.; stretched $\frac{3}{4}$-inch and recovered $\frac{1}{8}$-inch.

Fifth	"	"	"	"	$\frac{3}{4}$	"	"	$\frac{1}{8}$ "
Sixth	"	"	"	"	$\frac{3}{4}$	"	"	$\frac{3}{16}$ "
Seventh	"	"	"	"	$\frac{3}{4}$	"	"	$\frac{3}{16}$ "
Eighth	"	"	"	"	$\frac{3}{4}$	"	"	$\frac{3}{16}$ "

Ninth " " " broke in the eye, after a permanent stretch of 12 inches in 24 feet, and—in the eye of $\frac{3}{16}$-inch.

The next test was of round truss rods from M. B. Stotsinburg & Co.'s Works, Wilmington, Del., 1¾ inches in diameter, of Pencoyd Iron, made of selected scrap of the best quality, with an eye at one end and a screw at the other. The specifications as to strength were the same as in the former case. The eyes were made by turning the ends of the rods, and welding them to the body of the bar; and the screw ends were either upset or split, with a wedge-shaped piece welded in them, and swedged into shape so as to produce an enlargement that would enable the nuts to pass over the body of the rods; the nuts had a depth equal to the diameter of the upset ends of the rods. The testing machine was a hydraulic press, supplied by a force-pump with a one-inch plunger.

First application, 20,000 lbs. per square inch; stretched ⅛-inch in 14 feet, and recovered.

Second application, 24,000 lbs.: stretched $\frac{3}{16}$-inch and recovered.

Third	"	26,000	"	"	¼-inch		"	nearly.	
Fourth	"	50,000	"	"	7 inches	and	recovered	¼-inch.	
Fifth	"	55,000	"	"	9	"	"	¼	"
Sixth	"	52,000	"	"	11	"	"	¼	"

Seventh " 50,000 " broke in the body of the bar, near the eye, after a permanent stretch of 3½ feet in a length of 24 feet. The peculiarity in this last application was—that the bar stretched until it broke under a strain of 50,000 lbs., as fast as the force-pump could supply the power necessary to move the piston of the press with a pressure gradually running down below 50,000 lbs. Neither the eye or the screw received any injury from the tests to which the bar had been subjected. The important difference indicated by these tests in the two descriptions of iron—the one new and the other made from scrap, and both of the best quality of their kind—is that the elasticity of the former increased as it became gradually weakened by successive strains, whilst in the latter it decreased, although there was a greater capability of stretching before it finally gave way. They also indicate how emphatically the elastic strength, and not the breaking strength, should govern the use of iron in bridges, as each successive strain beyond that limit must weaken the bars and ultimately produce failure. It may also be doubted if, where the upsetting process is used—a process which, in accuracy and uniformity, is superior to all others for making eyes—there should not be an increased section round the eyes, to compensate for the derangement of the fibres produced by the operation.

Mr. McAlpine—How long was the test applied?

MR. STEELE—Ten or fifteen minutes, probably.

MR. COLLINGWOOD—In what sense is the term elasticity used ?

MR. STEELE—I use the term elasticity as indicating the power of iron to recover from any strain or stretch to which it may be subjected ; that power, the iron made from scrap seemed to have to a greater extent than the new iron, but the proportion of recovery to the stretch was the greatest in the new iron.

MR. COLLINGWOOD—The limits of elasticity certainly did not increase,—that is, the limits beyond which there was a permanent set, were not increased.

MR. STEELE—It seemed to be in this case ; but a single experiment is not sufficient to establish the rule as between two descriptions of iron, particularly as the proportion of recovery to the stretch, when strained beyond that limit, was the greatest in the new iron.

MR. C. SHALER SMITH—What was the diameter of the chord pins ?

MR. STEELE—They were all three inches in diameter.

MR. C. SHALER SMITH—Did you examine the fractures with a microscope to see if there were any signs of ruptured fibres from the upsetting?

MR. STEELE—I did not examine them sufficiently to indicate that; the iron was perfectly sound ; it broke when the leverage due to the shape of the eye obtained the greatest power, and there cannot be a doubt but there is a derangement of the fibres from the upsetting process. The fracture was on a direct line across the eye.

MR. MACDONALD—This is but another evidence of the importance of a proper pin connection ; the diameter of the pin seems to have been sufficiently large, but in proportioning the eye with 33 per cent. more material at the pin-hole than in the body of the bar, it is a difficult thing to insure a proper transfer of the strain; the practice at Phœnixville is to add 50 per cent. to the section of the bar for the strength of the eye. It has been found as the result of a number of experiments made in England, from 1862 to 1869, that by putting more material behind the pin, and easing the curvature where the eye begins, an increase of 25 per cent. in the eye gave the best results.

In upsetting by pressure it is difficult to graduate the movement of material, so as to fulfill these conditions. I think that Phœnix "Best Best" iron should stand more than 52,000 pounds, and the rupture in this case at the eye proved that the strength of the bar was not develloped, owing to a defective pin connection.

MR. BOLLER—We know that the transfer of the strain should be as

gradual as possible across the bearing point of the pin. Our practice is to make the radius of the connecting curve about double the diameter of the hub. I wish Mr. Steele had enlarged his experiments and taken a piece of the material from the body of the bar itself, and also from the hub, to see if there was any damage caused by the process of manufacture.

Mr. T. C. Clarke—I think the illustration is perhaps as good a one as this Convention could possibly have of the necessity existing for a series of experiments, which should be undertaken by somebody who would look at this matter from a purely scientific point of view. Each member can describe a few experiments that he has tried himself: there should be a series of experiments to show the strength and elasticity and other properties of American irons, the proper proportions and mode of manufacture.

Prof. Wood—It is known that a slight strain apparently produces a set in iron, which may be exceedingly small; but if produced by each successive strain, it may be affirmed that no iron structure is safe, because the set will increase.

You apply a strain to iron, say 10,000 pounds, and observe a permanent set; now, if you apply 10,000 pounds again, does all the elongation thus produced remain in the bar? So far as I have observed, it disappears or becomes indeterminable, after applying five or ten more strains. It has been a question in my mind, if we do not exceed a certain amount of strain after the iron has been submitted to an initial strain, whether it would not wholly recover itself.

Mr. C. Shaler Smith—I have made experiments testing up to 27,000 pounds, at which strain the elasticity was nearly destroyed. I have again subsequently tested the same piece of iron and obtained the same elastic point in the bar.

An experiment was made by Mr. Albert Fink on a bar from a bridge built by him on the Baltimore and Ohio R. R. The bridge was a thirty-feet deck span, located near the summit of the Alleghanies, and owing to the great increase in the weight of the engines found necessary on the mountain grades, the bar in question had been daily subjected to a strain of from 17,000 to 18,000 pounds per square inch. This bar, after having been six years in the bridge, was brought to Louisville in 1858, cut in two, and tested in comparison with a similar bar of the same grade of the same manufacturer, which had not been in use; the bar which had been in use broke at nearly the same strain as the other, 61,000 pounds; 61,500 pounds was the limit of the bar which had been in service.

THE CHAIR—How many applications of the load ?

MR. C. SHALER SMITH—The application of the load to the new bar was made gradual ; we were putting it on and taking it off at every 1,000 pounds increase of strain, to determine the point of elasticity.

THE CHAIR—What about the old bar ?

MR. C. SHALER SMITH—We cut it in two and tested each piece ; they broke very nearly at the same point.

THE CHAIR—Was that charcoal or anthracite iron ?

Mr. C. SHALER SMITH—That I cannot say.

THE CHAIR—It seems to be determined approximately, that in the bars experimented upon by Mr. Steele, the limit of elasticity of the new iron was superior to that of the scrap iron, it being about 24,000 pounds. As to the time at which the elastic limit of the bar would be impaired, the experiments related by Mr. Smith are very interesting ; the bar used six years was found to have an elastic limit of about 24,000 pounds. With reference to the eyes, the experiments seem to indicate that the cross-section opposite the pin ought to be 25 to 33 per cent. greater than the body of the bar. The Chair recommends to the special attention of the members of this Convention the suggestions of Mr. Clarke and Prof. Wood upon the subject of making careful experiments at an early date on the strength and elasticity of wrought-iron.

LVIII.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

TESTS OF BRIDGE IRONS.

A Paper by J. DUTTON STEELE, C. E., Member of the Society,

READ AT THE FIFTH ANNUAL CONVENTION, IN LOUISVILLE, KY.,
MAY 21ST AND 22D, 1873.

CHARLES HERMANY, C. E., IN THE CHAIR.

The first test was of chord links made of Phœnixville "Best Best," or double-refined new iron, 3 by 3 inches section, with upset eyes of the Phœnixville standard size, having a section round the eyes of 33 per cent. in excess of the section of the bar. The specifications required that the links should have an elastic strength of 24,000 lbs., and a breaking strength of 50,000 lbs. per square inch, with eyes strong enough to break the body of the bar. The testing machine was a lever with weights.

First application, 20,020 lbs. per square inch; stretched in 10 feet $\frac{1}{16}$-inch, and recovered.

Second application, 23,600 lbs. per square inch; stretched $\frac{3}{32}$-inch, and recovered.

Third application, 50,287 lbs. per square inch; stretched $\frac{7}{8}$-inch, and recovered $\frac{1}{8}$-inch.

Fourth application, 50,287 lbs.; stretched $\frac{3}{4}$-inch and recovered $\frac{1}{8}$-inch.

Fifth	"	"	"	"	$\frac{3}{4}$	"	"	$\frac{1}{8}$	"
Sixth	"	"	"	"	$\frac{3}{4}$	"	"	$\frac{3}{16}$	"
Seventh	"	"	"	"	$\frac{3}{4}$	"	"	$\frac{3}{16}$	"
Eighth	"	"	"	"	$\frac{3}{4}$	"	"	$\frac{3}{16}$	"

Ninth " " " broke in the eye, after a permanent stretch of 12 inches in 24 feet, and—in the eye of $\frac{3}{16}$-inch.

The next test was of round truss rods from M. B. Stotsinburg & Co.'s Works, Wilmington, Del., 1¼ inches in diameter, of Pencoyd Iron, made of selected scrap of the best quality, with an eye at one end and a screw at the other. The specifications as to strength were the same as in the former case. The eyes were made by turning the ends of the rods, and welding them to the body of the bar; and the screw ends were either upset or split, with a wedge-shaped piece welded in them, and swedged into shape so as to produce an enlargement that would enable the nuts to pass over the body of the rods; the nuts had a depth equal to the diameter of the upset ends of the rods. The testing machine was a hydraulic press, supplied by a force-pump with a one-inch plunger.

First application, 20,000 lbs. per square inch; stretched ⅛-inch in 14 feet, and recovered.

Second application, 24,000 lbs.: stretched $\frac{3}{16}$-inch and recovered.

Third	"	26,000	"	"	¼-inch		"	nearly.	
Fourth	"	50,000	"	"	7 inches	and	recovered	¼-inch.	
Fifth	"	55,000	"	"	9	"	"	¼	"
Sixth	"	52,000	"	"	11	"	"	⅛	"

Seventh " 50,000 " broke in the body of the bar, near the eye, after a permanent stretch of 3½ feet in a length of 24 feet. The peculiarity in this last application was—that the bar stretched until it broke under a strain of 50,000 lbs., as fast as the force-pump could supply the power necessary to move the piston of the press with a pressure gradually running down below 50,000 lbs. Neither the eye or the screw received any injury from the tests to which the bar had been subjected. The important difference indicated by these tests in the two descriptions of iron—the one new and the other made from scrap, and both of the best quality of their kind—is that the elasticity of the former increased as it became gradually weakened by successive strains, whilst in the latter it decreased, although there was a greater capability of stretching before it finally gave way. They also indicate how emphatically the elastic strength, and not the breaking strength, should govern the use of iron in bridges, as each successive strain beyond that limit must weaken the bars and ultimately produce failure. It may also be doubted if, where the upsetting process is used—a process which, in accuracy and uniformity, is superior to all others for making eyes—there should not be an increased section round the eyes, to compensate for the derangement of the fibres produced by the operation.

Mr. McAlpine—How long was the test applied?

MR. STEELE—Ten or fifteen minutes, probably.

MR. COLLINGWOOD—In what sense is the term elasticity used ?

MR. STEELE—I use the term elasticity as indicating the power of iron to recover from any strain or stretch to which it may be subjected ; that power, the iron made from scrap seemed to have to a greater extent than the new iron, but the proportion of recovery to the stretch was the greatest in the new iron.

MR. COLLINGWOOD—The limits of elasticity certainly did not increase,—that is, the limits beyond which there was a permanent set, were not increased.

MR. STEELE—It seemed to be in this case ; but a single experiment is not sufficient to establish the rule as between two descriptions of iron, particularly as the proportion of recovery to the stretch, when strained beyond that limit, was the greatest in the new iron.

MR. C. SHALER SMITH—What was the diameter of the chord pins ?

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LIX.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTES ON THE CRUSHING STRENGTH OF AMERICAN IRON.

A Paper by THOMAS C. CLARKE, C. E., Member of the Society,

READ AT THE FIFTH ANNUAL CONVENTION IN LOUISVILLE, KY., MAY 21ST AND 22ND, 1873.

CHARLES HERMANY, C. E., IN THE CHAIR.

In a Paper* read by the undersigned before the Annual Convention held at Chicago, June 5th and 6th, 1872, it was stated that certain experiments would be made at Phœnixville to determine the actual compressive strength of various forms of struts, of the full sizes and of the various shapes most commonly used in American iron bridges.

As such experiments on a large scale would involve a considerable expenditure of time and money, it became necessary to ascertain the accuracy of the methods commonly employed to measure the strains produced by a first-class press, capable of exerting a stress of 500 tons at least. The result of this investigation showed that there are no large presses where stresses can be measured with any considerable degree of accuracy.

A mercurial gauge which registers the pressure on the piston of a hydraulic press does not eliminate the variable quantity to be deducted from the registered pressure due to the friction of the leather packing against the sides of the cylinder. This varies in every press, and is greatest when the packing is new.

* XLII. Transactions.

The difficulty of ascertaining the exact pressure or tension on the specimen under test, has led the U. S. Government engineers, whose example has been followed by Kirkaldy, in England, to measure the stress at the opposite end of the specimen, by a system of levers. This answers well for moderate stresses ; but when we wish to employ stresses of 500 tons and over, the stress soon destroys the knife edges which form the fulcrum of the lever, and their friction invalidates the accuracy of this mode of measurement.

Investigation is now being made upon Emery's system of receiving the stress to be measured, directly upon a fluid contained under a flexible diaphragm. This pressure is transferred through a tube of extremely small bore to another diaphragm of say .01 of the area of the joint. The movement of the second diaphragm is registered by a very accurate system of levers, and the result recorded. Should this system prove to be as accurate in its results under heavy stress as it has under light ones, it is probable that new presses of much greater power than have ever before been used, will be constructed, and the proposed experiments then made.

A few experiments have been made upon the absolute crushing strength of Phœnix iron, which may be of interest, as giving considerably higher results than those upon English wrought-iron, reported by Hodgkinson.

The hydraulic press was in as good order as was possible. The packing had been in almost daily use for several months previous, and its friction was believed to be at a minimum. The mercury-gauge was compared and tested both before and after making the experiments.

According to Hodgkinson, "wrought-iron is crushed (*i. e.*, bulged) "with a compression of 16 tons (35,840 lbs.) per square inch; and when "the pressure exceeds 12 or 13 tons (26,880 lbs. to 29,120 lbs.), it cannot "in most cases be usefully employed, as it will sink to any degree, "though in hollow cylinders it will sometimes bear 15 to 16 tons per "square inch." (Report on Application of Iron to Railway Structures, p. 121.)

"When the length of wrought-iron tubular pillars does not exceed "10 or 20 times their width (*i. e.*, diameter), they fail by bulging or "widening of the plates, not by flexure of the pillar, and within this "limit the strength of the tube is nearly independent of its length." (Report, pp. 121, 163.)

"Of rectangular tubes of uniform thickness, none but those which had "thick plates compared with the width of the tube (not less than one-

"thirtieth) resisted 12 tons per square inch, but hollow cylinders gave "better results, some attaining the limit of 16 tons (35,840 lbs.) per square inch." (Report, p. 121.)

Thus, according to Hodgkinson, hollow cylinders of English wrought-iron whose length is so short as to yield by bulging or buckling instead of bending, and the thickness of whose plates does not go below one-thirtieth of the diameter, give the highest results, and will sometimes bear as high as 35,840 lbs. per square inch before yielding.

Six specimens prepared by sawing pieces, 4 inches and 8 inches long from small Phœnix columns, and facing the ends accurately in a lathe, were crushed, with the following results:

No.	Description.	Diameter over tube, inches.	Length, inches.	Ratio of length to diam.	Area, square in.	Total stress, pounds.	Pounds per sq. inch of area.	Thickness of metal.	Do in parts of diam.
1	4 in. regular A column.	$3\frac{31}{32}$	4	1	2,925	166,400	56,889	$\frac{5}{32}$	.04
2		$3\frac{31}{32}$	4	1	2,925	162,500	55,555	$\frac{5}{32}$	.04
3		$4\frac{11}{32}$	4	....	5,625	370,500	65,867	$\frac{5}{16}$	.07
4		$4\frac{11}{32}$	4	....	5,625	370,500	65,867	$\frac{5}{16}$	.045
5	4 in. regular B column.	$5\frac{1}{4}$	8	1.46	6,975	422,500	60,573	$\frac{1}{4}$	.045
6		$5\frac{1}{2}$	8	1.46	6,975	421,200	60,387	$\frac{1}{4}$	...
7	$2\frac{1}{2}$ in. round bar.......	$2\frac{1}{2}$	3	...	4 91	331,500	67,517	...	...

Mr. Collingwood—It has been stated to me, by Mr. Sellers, a gentleman of large experience, that the mercurial gauge for measuring high pressure is valueless, owing to a want of uniformity in the results. He stated, as the result of his experiment with an oil-tight cylinder, that there was in some way a variation which prevented him from obtaining correct results.

Mr. Steele—In former times we were in the habit of considering wrought iron as having much less compressive than tensile strength. These experiments indicate a compressive strength in the Phœnix columns about equal to the tensile strength of the iron, which strikes me as a very important general result. Another thing indicated by the experiments is, that the proportion between the diameter of the columns and the

thickness of the iron is about correct; with any other proportion I think the character of the rupture would have been different.

Mr. C. Shaler Smith—Referring to testing machines, my experience has been that it is far better to use machines with large cylinders and low pressure. The custom of using high pressure with a small piston is very largely prejudicial to the correctness of the experiment; but by using a lever and weight at the other end of the test bar, as a check to the pressure gauge on the cylinder, and by having so large a piston that the highest strains required in practice will not bring more than 18 or 20 tons per square inch in the cylinder, the strain on the spring of the gauge is kept within the limits of correct results, and the margin for error is greatly reduced.

Mr. Leverich—I will describe what is known as "Emery's" testing machine: imagine two hydraulic cylinders connected, each with pistons closely fitting, and yet without friction—the area of one piston being—say 100 times greater than the other—then any pressure communicated to the larger will be known when the pressure upon the smaller is measured—even if a very small movement of either piston takes place; it only being necessary that the fluid used be incompressible, and put in a state of tension.

Now, if the pistons be just removed from the cylinders so that their inner ends are coincident with the outer ends of the cylinders, and a strong flexible diaphragm interposed, it will be seen that each may move slightly without friction. This is the principle of Emery's testing machine. The strain upon a piece to be tested, is taken upon a large piston and communicated to a second and smaller piston, bearing such definite ratio to the first that its strain may be measured by the fluctuations of a mercurial column or the movements of a scale beam.

The details of the machine include some novel features; its use is subject to conditions of workmanship and operation which can alone be determined by continued trial.

Mr. Whipple—With reference to "Knife-edge" bearings or pivots for beams used in the measurements of forces, in the case of a boat-scale constructed by me, for weighing boats and cargoes upon the Erie Canal, and used in weighing from 100 to 300 tons at once, the whole weight is concentrated upon eight knife-edge bearings of eight inches in length each; this gives 64 lineal inches of knife-edge to sustain 300 tons; hence we readily see what these edges bear without essential inaccuracy. In weighing these loads a variation of 100 pounds is readily indicated.

Mr. Boller—Have these bearings been examined after usage, and how long had they been in use ?

Mr. Whipple—I had occasion to examine them after being used some thirty years; the original form of section was that of a right angle slightly blunted or rounded, and they had, by compression and wearing, come to have a bearing surface of about one-sixteenth of an inch in width. My theory is that where bearing surfaces are reduced to the minimum amount capable of supporting the incumbent load, the pressure must be nearly uniform over the bearing area, and may, without essential error, be regarded as all concentrated at the centre of gravity of that area ; hence it is not essential that the edge be perfectly sharp, but it may have a sensible width and the pressure be regarded as all taking place in the centre of that width, especially in case there be no rocking, or almost none upon the pivots.

Mr. Worthen—Col. Flad informs me he has compared his gauge with the mercurial column, and found that they agreed within one or two per cent.

LX.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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DETROIT RIVER TUNNEL.

A Paper by E. S. CHESBROUGH, C. E., Member of the Society,

READ AT THE FIFTH ANNUAL CONVENTION, IN LOUISVILLE, KY., MAY 21st and 22D, 1873.

CHARLES HERMANY, C. E., IN THE CHAIR.

At the date of the former Paper,* on this subject, read at the last Convention, the preliminary work on the Detroit River Tunnel was in a very encouraging state. The Detroit shaft had been sunk, and a drainage tunnel extended from it, for about 600 feet toward the Canada end. The Windsor shore shaft had been sunk to below the bottom of the drainage tunnel, which had progressed about 100 feet toward the Detroit end. With the exception of finding harder ground, and consequently making slower progress, than had been originally expected, the prospect of a successful completion of the work was brighter than at its inception, since previous to sinking the Detroit shaft there was a fear that very troublesome veins of water, supplied from the land and having a higher source than the river, might be met. For this reason the Detroit shaft was sunk first, as the borings on the Windsor side did not indicate such veins of water.

In the latter part of July, 1872, when the work on the Windsor end had progressed about 250 feet, through for the most part, very hard ground, some of which was blasted, a sudden irruption of sand and water occurred, which threatened to fill the tunnel out to the sump, and

* XLII. Transactions.

choke the pumps. To prevent this, a bulkhead was constructed near the face, but before it could be made sufficiently tight the workmen had to retreat some distance to make an apparently successful stand, and even this did not prove sufficient, so that a third and last bulkhead, still nearer the shaft, was put in. This state of things looked very discouraging, and it was, of course, impossible to tell the exact nature and extent of the source of the irruption, or how long it would continue. From the character of the water itself, as well as from other circumstances, it evidently did not come from the river, and there was reason to hope its flow would soon diminish. This hope was not disappointed, and about the 14th of August the face was again reached, the bulkhead (and 150 cubic yards of sand) having been removed. Regular operations were resumed, but after 30 feet of new tunnel had been built a fresh irruption of sand and water occurred, making it again necessary to put in bulkheads, and preventing further advance for four days more. By this time it was concluded that the source of the irruption must be a vein, and not merely a pocket of sand; still it was hoped that it might prove quite limited in extent, and soon be passed. On the 12th of September, after the work had been extended 47 feet further, a third irruption occurred. After another placing and removing of bulkheads, and taking out of sand, causing a delay of five days, regular operations were resumed, and 10 feet advance made, when a fourth irruption occurred.

By this time the contractors had become very much discouraged, and felt that to continue the drift on the same level would be ruinous to them, as the work was costing more than four times the price they received for it. Inasmuch as the work on the Detroit side had been extended about 1,200 feet—sufficiently far to drain the lowest portion of the main tunnel, and as the principal object now remaining was to explore the ground through which the main tunnel was to be built, it was decided to make a "lift-shaft" at the end of the drift on the Windsor side, and get into the ground through which it was proposed to construct the main work, thus avoiding, if possible, the irruptions which had become so troublesome. This was accordingly done, and a new drift started, at a level 10 feet higher than that of the drainage tunnel. The ground was much easier to excavate, but the irruptions, which formerly came from the top of the excavation, now came up through the bottom, there being a vein of sand at the level of the top of the lower drift. This was not a quicksand, nor usually running, and was only brought in, when it did come, by the force of running water. On reaching a point about 370

feet from the shore shaft, an irruption occurred, which continued so long that it seemed as if further progress in that direction was impracticable, in so small a drift, with the ordinary means of tunneling.

Before describing the further steps taken at this end of the tunnel, it will be well to mention what had been encountered and done on the Detroit side. The work was carried on there without any serious difficulty, and at a satisfactory rate of speed, until a point 1,110 feet from the shaft was reached. There the quantity of water coming from the bed-rock, immediately beneath, increased considerably. Gas had been more or less troublesome most of the way, sometimes making the men's eyes so sore that they had to quit work for a while. When a distance of about 1,180 feet had been reached, the machinery for ventilating the tunnel proved inadequate, and some delay was occasioned by having to put in more. Before the ventilating apparatus was started again, a man went out to the end of the work, and returned without having been injuriously affected by the air, which he said was bad. He reported a sand leak at the face. Two others then went out to stop the leak, which they expected to do in a few minutes; but they never returned alive. When they had remained as long as was thought necessary and did not return, the foreman sent a man to order them back, if their eyes were affected by the gas. He returned and said that they were dead. Others went in for them, but were unable to get them out alive, although one of them showed signs of life when first reached. (It was only after several attempts, and at great risk, that their bodies were recovered.) Previous to this, no one connected with the work had feared any fatal results from inhaling the gas, the greatest evil apprehended being sore eyes.

After the new ventilating apparatus was set in motion, regular operations were resumed, and the work was extended to a point 1,220 feet from the shaft. The influx of water here became so great as to require more powerful pumping machinery. It was thought best, however, not to require the contractors to incur this expense at the time, but to let them suspend work at this end until further developments were made at the Windsor side, where the prospect, as previously stated, was so discouraging.

At this juncture the contractors requested to be relieved from all further obligation to prosecute the work under their contract, which the directors agreed to, on conditions not necessary to mention here.

It was then determined to carry on the work at the Windsor end by the day, by means of two parallel trial drifts, and to begin the second one

at the shore shaft, at a level 10 feet above the grade of the drainage tunnel, leaving the latter to be used as a sand-holder in case of further irruptions. Thus it was hoped that, in either one or the other of the parallel drifts some progress might constantly be made; experience having shown that a stream of sand and water flowing into the tunnel at one point would never be accompanied by a troublesome one flowing in at another. In fact, it was observed that water which flowed from an orifice which at first discharged sand as well as water, ceased flowing either shortly before or just when a new irruption occurred at the face.

The upper drift, for a distance of about 380 feet from the shore shaft, was easily constructed, in some cases upwards of 20 feet of progress being made in 24 hours. This drift was continued to the right of the old one, beyond the lift-shaft, and no irruption occurred in it until an advance of about 20 feet was made beyond the face of the old or first drift. Then an irruption occurred, and the water and sand ceased flowing into the old drift, which was extended 50 feet, before the water returned to it and and left the new one free. The latter was in turn extended about the same distance, when the water changed over to it. Thus the work was carried on, alternately, in the old and new drifts, when the directors, becoming discouraged at its slow progress and excessive cost, ordered it stopped. The actual advance in new ground during the last two months was only 64 feet, and the cost about $7,500, or more than 6½ times the contract price.

Besides the discouragements connected with the work, the unusual severity of last winter caused such an interruption in the movement of freight across the river at Detroit, as to amount almost to "strangulation," certain and speedy relief from which was felt to be an absolute necessity; otherwise the already very heavy and constantly increasing business of the two railways interested must be largely diverted to other channels.

The decided refusal of the Canadian Parliament, a few years since, to grant a bridge charter has been succeeded recently by the granting of one to a company whose road crosses a few miles below Detroit, on condition that Congress shall grant one also. The matter is now the subject of investigation by United States Engineers, who are to report before the next meeting of Congress.

While the construction of the Detroit tunnel, as a simple engineering problem, cannot seem otherwise than practicable to the members of the profession, especially in the light of the experience gained on the Thames tunnel, and similar works completed since—the advisability of construct-

ing it, as a judicious expenditure of money, was left to be fully settled by the making of the drainage tunnel. The engineer believed, from the original borings, and from the earlier operations on the drainage tunnel, that the main work was not only practicable but advisable ; later developments, however, throw much doubt upon its advisability.

It remains to answer several questions which will very naturally occur to members of the Society, such as—

1st. Why was not the character of the veins of sand, which gave so much trouble, revealed by the borings made before the work was begun ?

The borings did frequently pass through small deposits of sand, but pockets of this material are so common in drift clay, that nothing is thought of them in ordinary tunneling. As already mentioned, fears were entertained that trouble from a great influx of water might be encountered in the Detroit end, but no such difficulty occurred there.

2d. Why could not the orifices through which the irruptions occurred be stopped ?

This experiment was tried several times, but it always ended in making matters worse instead of better. If the influx was stopped at one point, it broke out at another. If the whole face was completely protected against it, the fresh joints in the masonry would be washed out. This will not be wondered at, when it is stated that the source of the impressing water was ascertained, after the stoppage of the work, to be more than 100 feet above the bottom of the drainage tunnel.

3d. Why could not a shield have been used to advantage ?

This was thought of, but experience, both in Chicago and elsewhere, had shown that shields, in such small drifts, through soft clay, are exceedingly difficult to keep in line. Such would have been especially the case on this work, where, after each irruption, the end of the masonry and, toward the last, the timbering, were so twisted and broken, laterally and vertically, as to require rebuilding in several intances.

4th. Could not the work have been carried on by the pneumatic process ?

Besides the fact that no horizontal drift of any length is known to have been made in this manner, it will be sufficient to state, to those familiar with this process, that work executed under a pressure equal to 90 or 100 feet head of water is not only very expensive, but hazardous to human life.

Another reason for not excluding the sand permanently, if it could be

done, was that by letting it come in till it ceased to flow of itself, the ground would be left in a much better state for the main work. This belief was confirmed by making the second and parallel drift, in which no irruption occurred until after all the old ground worked in had been passed.

A MEMBER—How was it ascertained that the source of the water was 100 feet higher than the tunnel?

MR. CHESBROUGH—The top of the shore shafts are 106 feet above the bottom of the drainage tunnel; after the work ceased entirely, the shaft was allowed to fill up, when the water rose to the top, and now flows over.

MR. MCALPINE—It is stated that the inflow of the water in these tunnels evidently came, not from the river, but from a higher source. When building the dry docks at Brooklyn, precisely the same result took place. We were 40 feet below the level of tide-water, and fresh water came in with a head of 50 feet—perhaps 100 feet—higher than that of the salt water.

MR. C. SHALER SMITH—Did this irruption come from below?

MR. CHESBROUGH—I am sorry I have not a section here. I am satisfied that it came from below. There was a vein of sand just above the top of the drainage tunnel, and when there was an irruption it seemed to come from that. The irruptions at first were rather from above, then we went up and obtained a rock grade on the Windsor end, and *there* the water pressed upward, and I think it came from the rock. I will give you another reason: at a place called Sandwich, below Detroit, where there are sulphur springs, the water frequently rises 30 or 40 feet above the river.

MR. C. SHALER SMITH—Almost the same circumstance happened in sinking one of the piers of the St. Charles Bridge by the pneumatic process. At about 50 feet below the surface, a sulphur spring was struck, which, from the manner in which the water rose in the caisson, had evidently a greater head than the river above it.

MR. CHESBROUGH—I have heard of several instances like that.

MR. BOLLER—You spoke of sulphur springs in the neighborhood, and suppose that the water which came in on you had a similar source; was it sulphur water?

MR. CHESBROUGH—Yes, sir; it was.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

LXI.

A NOTE ON THE RESISTANCE OF MATERIALS.

By Prof. ROBERT H. THURSTON, Member of the Society,

READ AT THE REGULAR MEETING, NOVEMBER 19TH, 1873.

On the 13th ultimo, an apparatus for determining the torsional resistance of materials, which I had designed for use, in illustration of my course of instruction, and to which I had fitted an automatic recording attachment, was exhibited to the National Academy of Science, at the late session held at this place, for the purpose of showing the peculiar adaptability of the machine for the determination and analysis of the action of physical and molecular forces in resisting stress, and to illustrate the bearing of experiments already made upon scientific investigations of molecular relations.

At the close of the meeting, a test piece of wrought-iron was left in the machine, exposed to a strain which had passed the limit of elasticity, and with a distortion of 45 degrees, the intention being to determine whether, as has been suspected by some writers and by many engineers, "viscosity" is a property of solids, whether a "flow of solids"* could occur under long-continued strain just equilibrating, when first applied, the resisting power of the material, or whether the "polarity" of Professor Henry is an absolutely unrelaxing force.

The metal was left under strain twenty-four hours, and had not then yielded in the slightest degree. This result, and the results of other similar experiments since made confirming it, indicates, that metal strained far beyond the limit of elasticity, as above described, does not lose its power of resisting unintermitted static stress.

The important bearing of this fact upon the availability of iron, and of steel, which also behaves similarly, for use in constructions exposed to severe strains, is readily seen.

* Mon. H. Tresca; Sur l'Ecoulement des Corps Solides. Paris, 1869–72.

After noting the result obtained as stated, it was attempted to still further distort the test piece, when the unexpected discovery was made that its resisting power was greater than when left the previous day, an increase of resistance being recorded amounting to about 25 per cent. of the maximum registered the preceding day, and approximating closely to the ultimate resistance of the material. Repeated experiments, continued up to the date of writing, confirm the following previously undemonstrated principle; that iron and steel, if strained beyond the limit of elasticity, and left under the action of the distorting force which has been found just capable of equilibrating their power of resistance, gain resisting power to a degree which has a limit in amount, approximating closely, if not coinciding with the ultimate resistance of the material, and which had a limit, as to time, in experiments hitherto made, of three or four days.

Releasing the piece entirely and again submitting it to the same force immediately, does not produce this strengthening action.

There is some evidence, that is confirmed by theoretical dynamic principles, that the increase of strength noted is not accompanied by a change of resilience, but that the gain of resisting power is at the expense of a proportional amount of ductility.

The diagrams obtained during this research will be presented at a future time, when the investigation shall have been completed.

The interest and importance attaching to the discovery of the principles above enunciated, to our profession as well as to science, will, I hope, justify the presentation of this note.

LXII.

THE PRODUCTION OF TRAFFIC AND THE TRANSPORTATION OF FREIGHT AND PASSENGERS.

A Paper by MARTIN CORYELL, C. E., Member of the Society.

READ AT THE ANNUAL CONVENTION HELD IN LOUISVILLE, KY., MAY 21ST AND 22D, 1873.

CHARLES HERMANY, C. E., IN THE CHAIR.

Early history and the ruins of ancient civilization attest that Asia and art of Africa were first densely populated; that agriculture, com-

merce and the arts were developed in high degree, and that the carriage and exchange of products of the soil, materials for building, munitions of war, and the objects of commerce then were necessary to a luxurious people. Among the remains of that age, there is nothing to show the use of modern substitutes for manual labor or beasts of burden, in performing the work of inland transportation.

China and Egypt had canals at an early day, and used their rivers similarly for travel and transportation. Greece and Rome employed ships in commerce and for war, constructed canals, opened roads, and erected bridges, the remains of which are still to be seen.

Coming down to modern times, the first canal in France—between the rivers Loire and Seine, 34½ miles long—was begun in 1605, and completed thirty-seven years afterward; in England, the first—the Bridgewater Canal—was begun in 1758. Coaches were introduced about 1605, prohibited in 1635, and in 1770 only 1,000 were registered in the whole kingdom. With coaches, turnpikes, or roads on which tolls were levied, came into use; about 1790, Telford and McAdam began improving them, and in 1819 Parliament appointed a committee to examine into their condition.

In this country—soon after the Revolutionary war, as agriculture, manufactures and commerce began to thrive, the people, and particularly those on the seaboard, saw the need of a ready means whereby the products of the soil and the forest could be exchanged for commodities brought over the seas or made up in town and city. The experience of older countries was consulted, and turnpikes were determined on, not only as meeting a want much felt, but as a profitable investment for surplus funds; charters therefore were granted by State authority, and soon roads were built, connecting growing cities with fertile agricultural regions or thriving manufacturing centers.

Pennsylvania was the first State to have regularly a graded and stoned turnpike, which was chartered in 1792 and finished in two years; it was from Philadelphia to Lancaster, 62 miles long, and cost $465,000, or $7,500 per mile. So beneficial were its effects upon trade, and the country through which it passed, that it was extended to Pittsburgh, the most westerly town at that period, 303 miles distant, with ascents not exceeding 3¾ degrees, and passengers were carried in post-coaches from there to Philadelphia in 60 hours. When the road was completed, the traffic was not sufficient to pay dividends and for repairs, but in a few years it so increased that double loads were carried in vehicles drawn by

4, 6, or 8 horses, and consequently the tires of the wheels of the "Conestoga wagons" were widened from $1\frac{1}{2}$ to 5 or more inches. The results were that, under the effects of frost, rain, and the heavy traffic, turnpikes became expensive and unprofitable; and people, in good weather, would travel over a parallel road to avoid paying toll. Soon they were unpopular and condemned by the community generally, and most unjustly; for, until over-burdened by the quantity and weight of traffic, they served well their purpose; thereby cities and towns were built up, the lands through which they passed were increased in value fivefold, and the general enhancement of taxable property was ten times the original cost of the turnpikes. Now that they are relieved from heavy and ruinous loads, they are, if well made and kept, as useful and profitable as in their early career.

Canals were next in this country to relieve commerce, by an exchange of productions, and in their construction New York was first, with that ever-enduring monument of enterprise, the Erie Canal, which was the model, and furnished the engineers and contractors for many others throughout the various States. The history of canals is similar to that of turnpikes; the investments made in them were, in a great measure, lost to the original projectors and owners; generally they were located along rivers, subject to injury by extraordinary freshets, and extended so far towards the river's source that the traffic necessarily done in seven months only of the whole year was made unprofitable by the amount of lockage, scarcity of water, and effects of frost.

When the early canals were completed, their capacity for transportation was so far in advance of the needs of the country, that they largely absorbed the traffic of turnpikes without, for years, earning enough to defray expenses and pay dividends; but as it was with turnpikes, wherever canals were located, manufactures, trade and travel greatly increased, cities and towns were built up, and the adjacent country became prosperous and wealthy; consequently, in time the canals were overcrowded with the trade they had reared and were unable to accommodate, and soon, therefore, they too were, in degree, condemned and neglected.

The railroad next claimed public attention for the carriage of freight and passengers, and when the locomotive was made its special adjunct by George Stephenson, it took rank and character which led to a success that completely revolutionized transportation and its various interests. Like the turnpike and canal, the railroad in early years was subject to trials of poverty, want of trade and depression of stock, but now it is

triumphant, fearing only that the telegraph may take from it the postal and money business of the country as effectually as it did from its slower predecessors.

At first, railroads were only intended for portages between navigable waters; it was generally conceded that they would for many years be all-sufficient when the rivers and lakes of the great West were reached, and it was deemed useless and impracticable to think of proceeding further without transhipment. The productions of the South and West so accumulated that the labor and delay at points of transhipment became a serious cause of waste and cost, which, added to the cost of transportation, tended to exclude these commodities from Eastern markets.

The scheme of bridging the Ohio, which floats a vast and increasing commerce in fleets of boats of every style, was violently opposed by the Federal government and the people at large, in Congress and the Courts, to prevent an attempt characterized as chimerical. The history of the Wheeling bridge attests what a grand success it was; it does, and may it ever stand, a pioneer monument of commercial enterprise and engineering skill, proving that traffic by rail need not destroy or greatly obstruct traffic by water. When a few more bridges, as the St. Louis and East River, and a few more tunnels, as the Detroit and Hoosac Mountain, are completed, the chief problems in railroad construction will be solved, and the question "Will it pay?" can be answered.

One of the earliest and boldest innovations of the railroad upon water navigation was when a line was projected and built along the Hudson river. Our worthy and Honorary Member Mr. John B. Jervis, as engineer, demonstrated that the Hudson River Railroad was not only practicable to construct, but profitable to operate. Most people thought it a wild scheme thus to contend for the trade of a free and noble river like the Hudson, with gorgeous steamboats, moving 16 miles per hour, numbers of sailing vessels and acres of floating timber; all without the least tax for tolls, repairs or management, while transportation by railroad would be subject to the large expenses of construction and operation. The result is before us: now, two double track railroads, well equipped and prosperous, share the traffic with the river—a third is projected, and there is trade enough for all; not to mention the vast consequent improvement of property and increase in values, along these lines, and as far as their connections extend.

Mr. Asa Whitney, formerly of New York, now of Philadelphia, conceived the "audacious project" of constructing a railroad, thousands of

miles in length, from ocean to ocean, through a wild and mountainous region, not only for traffic but to build up cities, develop the country, and divert trade from what had been always considered its natural channel, the sea, across an uninhabited continent. Heretofore commerce had directed the means of transportation; now the railroad was to win commerce from nations where before it had scarcely existed. In May, 1849, Mr. Whitney published a pamphlet entitled "A project for a railroad to the Pacific," in which were set forth many of the details of the grand scheme as it was afterwards accomplished. The road has been built for years, and yet his name is rarely mentioned in connection with this, one of the greatest undertakings of modern times. Now the North Pacific and the Southern Pacific Railroads are in course of construction, other lines, through British America and Mexico, are projected; all to compete for the traffic of Asia and the Pacific Islands, and to aid in peopling the large uninhabited areas of this continent.

In contrast to these broad plans for commercial and national progress, attention is called, without comment, to the following extract from a letter of Hon. Jefferson Davis, Secretary of War, dated February 24th, 1857: "Under the appropriation of $30,000, made on the 3d of March, 1855, seventy-five camels have been imported. The limited trial which has been made, has fully realized my expectations, and has increased my confidence in the success of the experiment."

Cheap, or as they are commonly termed, "narrow gauge" railroads, with light rails and well-constructed machinery will in time, be substituted for common roads, as feeders and distributers of main lines. In populous manufacturing or in agricultural districts, where the traffic is insufficient for more expensive lines, these may take it at profitable rates until roads of greater capacity are required.

Although railroads and their machinery, as a means of transportation, seem to be nearly perfected, the studies and services of the engineer are not soon to be dispensed with. A brief consideration of subjects now more or less prominent show rather that the field of his labors will in the future be more extended and diversified; a few of these subjects I will mention.

Marsh and swamp lands are to be reclaimed by embankments, ditching and subsoil drainage; rice, cranberries, cotton and other similar crops are to be irrigated; lands are to be cultivated and their products gathered by the aid of steam machinery; forests are to be preserved from the ravages of fire, and extended over barren lands, to keep up and

increase the supply of fuel and timber, and to equalize the amount of rainfall.

The water in our streams is to be stored up in time of plenty, for domestic and sanitary purposes, to extinguish fires, irrigate lands, furnish power, and to maintain the navigation of canals and rivers. Rivers are to be improved and secured; basins, docks and wharfs for shipping, elevators for grain, and warehouses for merchandise are to be constructed. Canals are to be reconstructed, an abundance of water for increased traffic provided, and the locks extended to take in without delay, at one time, a steam-tug and the boats she may have in tow.

Railroads are to connect the large cities of this continent, located, equipped and operated to maintain a speed of 100 miles per hour with safety and certainty. Superior and intelligent management must reduce the rates of freight so that the products of the soil and the mine, in regions widely separated, may be exchanged without loss, and at a fair, remuneration to the producer.

Safe and permanent buildings, for commercial purposes, with foundaations "strong and deep," are to be erected capable of resisting fire, floods, and the severities of our climate, fitted with machinery for hoisting and moving goods or grain, well ventilated and thoroughly adapted to the secure preservation of the property contained, without danger from loss.

One feature more or less relating to transportation, connected with the mining and marketing of coal, and in a less degree with the handling of ores and the manufacture of iron, has long exercised those managing these interests. The trade demand for coal in special sizes is irregular and spasmodic, whereas its production and preparation by suitable machinery is uniform, day by day, and in such quantities that accumulation of any particular size soon stops the entire work. In early spring there is a large demand for "lump" and "steamboat" coal, and but little for "prepared" or "special" sizes, which hence have for months to be forced upon the market at ruinous prices, or wasted on the "culm" or dirt-pile, to make room and keep the "breakers" clear. In the fall and winter this is reversed; orders then come in so fast for "prepared" sizes, that extra machinery is employed to crush the best coal and thus supply the demand at an enormous expense and waste, most of which in the end has to be borne by the consumer.

This loss and more could be saved if proper arrangements were made to transport daily to market, direct, the entire production of the mines,

and there have storing grounds and the necessary appliances to prepare the material as required for immediate use, so that which is wasted at the mines could be utilized as cheap and useful fuel. More than one-half of the coal in our mines is wasted there or in preparation for market; the mountains of coal-dirt which disfigure our coal districts should cause regret and shame to the capitalist, engineer and miner. All the items that enter into this waste can be taken up and adjusted, so that in the anthracite coal trade alone fifty cents per ton, or ten million dollars annually, may be saved.

The present equipment of railroads—particularly of those specially devoted to the coal trade—is in design and build adapted to but a limited variety of traffic, and consequently cars are returned empty, or when rolling stock is in great demand, stand for months upon a siding, depreciating in value more rapidly than when used. By the system of car construction, and the want of adaptation to required uses, toll rates are necessarily increased; and in consequence of this and other abuses, it will only pay to send the choicest and most valuable products to their natural market or place of consumption.

All these things are within the province of the Civil Engineer; but no one man can singly effect their improvement. It is the aim and end of this Society to include within its membership those who not only can lay foundations under rivers and seas, tunnel beneath lakes and through mountains, span navigable waters, and unite the extremities of a continent, but who, because of their integrity and experience, possess the confidence of those who supply the means, as well as of those who do the work of these undertakings. With such in cooperation, our profession can always perform whatever trade, commerce or the good of our common country may require of it.

Mr. McAlpine—On the 8th of May, at the invitation of the Chamber of Commerce of New York, I addressed the merchants of the city on the subject of transportation,* a very broad field. Certain results of investigation made in preparing that address may perhaps surprise you, as they did me.

Corn sometimes is burned for fuel in the West, simply for the want

* Address by Hon. William J. McAlpine before the Chamber of Commerce, at the Cooper Union, on the Extent of the Products of the food producing Interior of the United States, the Channels of Transport to Market, their relative Capacity and Economy, what Improvements or new Routes may be made to increase and cheapen Transport, and how these may be made conducive to the Interests of the Merchants and Citizens of New York City, May 8th, 1873.

of transportation. I compared the cereal production of the United States for the last three or four years with that of the civilized world elsewhere. The cereal production of the world, in bushels, is a little short of five thousand millions; the United States last year produced fifteen hundred millions, of which ten Western States, including Kentucky, produced a thousand millions, or one-fifth of that of the world. I became satisfied that two million tons of the production of the great West fail to reach the Atlantic markets, worth, including transportation, at least two hundred million dollars; in other words—and because every dollar's worth of production sent to the seaboard buys another dollar's worth that is returned to the country—the Atlantic markets have lost four hundred million dollars, and the people of the West one-half that sum.

Referring to the question of difference between communication by rail and by water, I ascertained that the four trunk railways from the East to the West—the New York Central, the Erie, the Pennsylvania, and the Baltimore and Ohio, carried only one-fourth of all that came East; the remainder came over the Erie and Welland canals. In regard to the cost, the products of the country may be carried 1,000 miles by water almost as cheaply—as 100 miles by land. Freights from Albany to New York are the same as from New York to Liverpool. From New York to San Francisco the rates per ton are—by the Pacific railroad, about 3,000 miles, from $60 to $100; by the Isthmus of Panama, about 7,000 miles, $20, and around Cape Horn, about 16,000 miles, but $12.

Railroads and canals are not antagonistic, they are parts of the same system and both indispensable. Railroads are required for the carriage of passengers, valuable parcels and perishable articles; but in this country they cannot be substituted for water conveyance. Who expects to see in his day the Erie Canal abandoned? The best talent and skill has been engaged in the management of railroads, which is not the case to any considerable degree with canals. To-day, steam has not yet been successfully tried on the Erie Canal, and boats are still towed by horses, as they were by the Ptolemies in Egypt three thousand years ago. The ocean steamers have not yet taken the place of sailing vessels. Thirty-five or forty years ago the locomotive was imperfect, and its success was uncertain; it has been completed within a few years, comparatively, and now is the greatest machine of the age; the combined achievement of the superior intellects applied to its development.

Prof. Greene—I have given the subject of steam canal navigation

some attention for the past two years, having been connected with a commission in New York, appointed to test the various plans proposed for introducing steam upon the Erie Canal. My estimates show that the cost, including all expenses, of moving boats by horse-power in the canal and on the Hudson river is about 5 mills per ton per mile. The cost of transportation by rail depends upon the grade, expense of construction and many other circumstances. On the principal trunk lines the cost at present seems to be about 1¼ cents per ton per mile. I have made estimates to ascertain how far this might be reduced on lines devoted exclusively to the transportation of freight, where trains were moved without interruption or being laid up on side-tracks, and found that the minimum cost, including all items of expenditure, would be about 7¼ mills per ton per mile. The experiments made with steam upon the canals during the last two years show a reduction of cost, as compared with horse-power, of from 22 to 25 per cent.—based upon transportation from Buffalo to New York, by way of the Erie Canal and Hudson river. If steamers were used exclusively, this saving might possibly reach 50 per cent.

I have considered the different ways of towing. By the ordinary canal-boat, carrying its own machinery with 200 tons freight, and towing two other boats of the same size carrying from 225 to 235 tons, the cost seems to be about the same as by that where the machinery is applied directly to each boat; the objection is that boats towed at the speed of 3 miles per hour become unmanageable, and captains will not take the consequent risk; even at 2½ miles per hour there is danger of injury by collision.

Where a tug-boat tows two or three loaded boats the cost is decidedly greater than if each boat carried its own machinery. The reasons are obvious; the tug-boat would at first cost as much as three or four ordinary canal boats. I cannot quote the figures, and will only suggest a line of investigation, whereby any person interested in this subject may arrive at a conclusion. Another reason is found in the application of the power. The size of the boat is determined by the size of the load. A single boat may carry and use as much propelling power as is required by those making up the tow; but there is always a certain loss from slip, which increases rapidly with the number of boats towed and the power expended, so that when three or four boats are towed, the loss may be 50, 60 and even 80 per cent.—that is, the power utilized may be only 20 per cent. instead of 50 to 70 per cent., as it should be.

Another objection to moving boats in tow is that much time is lost in lockage. A single boat may pass a lock in about five minutes; where

there are two boats, one being towed after the other, the first will go through as quickly as a single one, but the second boat, having no power, will be twice as long, and the time thus lost will similarly increase for every additional boat towed, so that when there are four or five boats, a large part of the time required to make the trip will be spent at the locks, where the boatmen are idle, the power is useless, the fires are burning and the interest account is going on. Then again, a towed boat, having no power of its own, will not last as long as a boat having its own power, for the reason that it cannot avoid collisions.

I have given attention to the Belgian system, and would say, that while it is perhaps successful in Belgium, it could not be so here. It consists of a wire cable, laid in the bottom of the canal, which works over a clip dam operated by machinery. I understand that the Belgian canals, for the most part, are very straight and have few locks; on the contrary, American canals have many sharp curves and excessive lockage, both of which are serious obstacles to the successful introduction of such a system.

Mr. Shinn—I do not know that I can add any very precise information to that already possessed by the Convention on this subject, but I will make a few statements which, while they differ in some respects from those of gentlemen who precede me, may open a field for investigation. This subject of transportion is the question of the hour. During the past six months it has largely entered into the debates of Congress, occupied the time of State Legislatures, and been discussed in the meetings of farmers' granges, and by other assemblages. There is a demand to know whether transportation cannot be performed better and cheaper. Investigation has naturally caused discussion, and a great variety of views are put forth, most of which do not proceed from a scientific or professional source, or present facts which can be relied on to determine the real state of the question.

It has been said in this Convention that in this country railroads have received the attention of the best engineering minds—not in the broad sense, that to construct and maintain them the ablest engineers have in some capacity assisted, but that in railway management they have largely taken part. I think this is an error; surely those who are prominent in managing the principal railways are not noted as skillful engineers, but rather as successful manipulators of legislatures, operators on Wall street, speculators who get up corners in stock and increase or depreciate nominal values. These are the men who control railroads—not by virtue of their

ability as practical managers, but because they own stock, or possess the confidence of those who do. No men in this country are less likely to become either manipulators of legislatures or victims of Credit Mobilier schemes than those of our profession; prominent engineers do not get up schemes of that kind either to run railroads or to build them. On the other hand, men who control and operate our railroads are not investigating the scientific basis on which the economy of transportation depends, or employing others to do so, and as a consequence the wildest notions prevail on the subject.

It is asserted by those who have looked into the matter somewhat, that it costs $1\frac{1}{2}$ cents per ton per mile to transport freight by rail. I am prepared to say that on the more important railroads of this country freight was transported at a profit last year for $\frac{7}{8}$ cent per ton per mile, and therefore the actual cost must have been less. The rate in 1872 for the transportation of fourth-class freight from Chicago to New York, was about 40 cents per 100 pounds, of which 5 cents was expended at New York to get it across the river and deliver it to vessels in the harbor; leaving 35 cents per 100 pounds, or $7.00 per ton received for transportation over 900 to 960 miles of railway. The roads which carried freight at this rate never made more money than last year, hence this business was not done at a loss, as it was a large proportion of the traffic. From this it is evident that the maximum cost was 7 mills per ton per mile; that, of course, was upon freight moved in full trains of loaded cars, and under circumstances most favorable to the present railway management.

Instead of the economy of railway transportation receiving the attention of the best engineering talent in the country, it seems to me that such talent has been studiously avoided, and rigorously excluded from any controlling voice in the management. Within my own knowledge, roads for years have been run with an equipment which was not more than two-thirds adequate to do the business pressing upon them. In this case the railway was a machine capable of doing twice or three times the business it was doing, but its equipment was insufficient to employ it more than one-third or one-half the time. In one instance, after frequent representations of this fact to the managers, an unusual pressure of business having convinced them that more equipment was needed, they suddenly increased it without making any corresponding extension of terminal facilities or side tracks, and then the machine was disproportioned the other way. There was nearly, perhaps quite enough

equipment for the time, but sufficient terminal facilities to dispose of the traffic as fast as it arrived, and side tracks to allow the trains to pass without delay were lacking, and so the business of the road was crippled by blockades at the terminus, and along the line. Where 400 cars per day should have been handled, not more than 100 got through; thus illustrating, in not an encouraging manner, the kind of engineering talent applied to some of our best railroads.

The capacity of railroads to do business, compared with canals, should not be judged by the amount done upon the four existing trunk lines, none of which, so far as I know, are managed upon a scientific basis. They are not generally controlled by men who value engineers, or employ them to do anything more than to locate the road, and, perhaps, construct some of the works. Hence these lines have been slow to develop the resources of the country, procure sufficient equipment, acquire necessary terminal facilities and adequate means at tide-water to dispose of what freight could be brought there. For instance, the Pennsylvania Railroad, which has the reputation of being one of the best managed railroads in the country, has been unable to get rid of more than 60 cars of grain, in bulk, per day, without being blocked up. The number of cars is not limited until they are really blocked, and then the blockade is transferred back from Jersey City to Philadelphia, and from Philadelphia to Pittsburgh, until every yard on the line from Jersey City to Chicago is full. The need of increased facilities at Jersey City has been for some time very evident, but the stockholders of the Camden and Amboy Railroad declined to spend four million dollars there to get rid of freight for the Pennsylvania Railroad, which they hauled only 90 miles. While the latter road would be greatly benefited by an expenditure of this sum in filling up docks, increasing depot facilities and building elevators, it manifestly would make no return to them, and therefore they would not appropriate it. This particular instance of a single defect in railway management will in due time be remedied.

It is true that a vast amount of agricultural products is not taken to the seaboard, but without doubt a much larger amount could be carried by existing roads if prices obtainable for it there would justify its shipment. Whether the price of these products can be reduced is mainly a question whether the cost of transportation can be lessened. As I have stated, in one case during the last year, this cost to the consumer, and not the railroad company, was from 7 to 8 mills per ton per mile, which I believe can be considerably reduced.

Take, for instance, the item of dead weight: A common well-built country wagon, weighing about 800 pounds, will carry 3,000 on any fair country road, and without injury pass over obstructions which cause it to fall one, two or more inches, the paying weight being about 79 per cent. of the whole. The ordinary box-car in use upon our railways at the present time weighs about 10 tons; its maximum load is generally about 11 tons, while its average load is about 8 tons; the paying weight being from 44 to 52 per cent. of the whole. It does not seem reasonable that the weight of a car constructed to run upon a smooth even track, without a fall, should be so disproportioned to the load carried. The proper relation and proportion to each other of the different parts of railway machinery is a subject worthy of consideration by the best engineers.

What amount of tonnage may be carried over a railroad is, to a great extent, a matter of speculation; but if, as has been already projected, a double-track railroad were exclusively used for a freight traffic, with its trains always running at a uniform speed, without turning out or laying over, it is manifest that many trains could be moved on the two tracks. The ordinary rule is for freight trains to follow each other five minutes apart, but by the English block system, or a similar one, this may be reduced to two minutes. If ten minutes are allowed between trains, and the cars are of construction and engines of power so that each train will carry 200 tons (more than which is done on many railroads now), 1,200 tons would pass in one direction every hour. 1,000 tons per hour would be 24,000 tons a day, and 7,200,000 tons in a year of 300 working days, which is much more than was carried last year by the four trunk lines between the East and the West, as stated in this discussion. In an engineering point of view, there is nothing improbable in that. I will not attempt to compare the cost of carrying seven millions of tons on one railway with that of carrying two millions of tons on four; evidently it would be much below the least sum already stated as the cost on one, if not all, of these four first-class railroads, namely, 7 mills per ton per mile.

A short time ago the "Pittsburgh Commercial," a paper that generally presents enlightened views, "in an article on this subject of transportation," suggested that if a railroad were devoted exclusively to freight traffic, and operated by parties who owned and run the cars they used (and by inference the locomotives also, although it was not so stated), the same as boats are now owned and run on the canals, the present cost to the consumer would be reduced three-fourths. Cereals grown in

Iowa, Kentucky. Missouri, and other Western States, are transported to the seaboard, at an average cost for the whole year less than one cent per ton per mile, therefore this article assumed that it could be done for 2½ or 3 mills. Statements so far from fact fail to engage the attention of transporters, and lead to no good result.

A complete and authoritative investigation into the cost of transportation, made by intelligent parties, capable of deducing principles from established data, would not only be very instructive to engineers and railway managers, but of vast importance to the country at large. There is nothing of the kind now, nor can I suggest the source from whence any authoritative statement may be expected, unless it is this Society.

Mr. Rothwell—In regard to the utilization of waste coal, alluded to in the paper read, it may be said that engineers and railroad men are more interested in the consumption of coal than in the details of mining. What the quantity of that consumption shall be, depends much upon the cost of coal where it is used, which of course varies with the charges for freight, the amount of waste and the expense of mining.

The waste of coal occurs either in mining, preparation for market, or transportation and storage. My estimates of the quantity lost in mining mostly refer to anthracite coal, though they would apply to the bituminous coal of this country; the two being generally mined on the same system—that of chambers and pillars, whereby a portion of the coal is taken out in rooms, and the remainder left to sustain the roof. In mines worked at moderate depths, up to 300 or 400 feet, and where consequently the pressure of the overlying strata is small, not less than 25 per cent. of the coal is left in pillars; probably 33 per cent. will nearly represent the average in mines of various depths.

A great portion of the 67 per cent. taken out is broken into sizes to suit the market. This has been done so long that consumers expect it, although usually the coal could be burned in lumps or broken where used, and the resulting waste avoided; for which, as well as the cost of breaking at the mines, the consumer, of course, has to pay. In preparing coal for market—that is, breaking, screening and washing it—the waste varies from 15 to 45 per cent.; where crushers with sharp steel teeth, and other suitable machinery are used, this is much reduced, and perhaps 25 per cent. may be taken as an average loss from this cause. The fine coal in the anthracite region is thrown aside as useless. Attempts have been made to manufacture it into a compressed fuel, which was found to cost more than the coal in lumps.

Then there is the waste in transportation. Most of the coal cars used for transporting anthracite coal have high bodies, whence the shutes from which they are loaded are about 8½ feet above the track; the coal falls from them to the bottom of the car, and consequently a portion of it is broken. It is then carried at considerable speed, especially where there is a down grade, over the roads, some of which are pretty rough. Experience shows that the breakage varies very much with the speed of the trains; in bituminous coal it is more apparent than in anthracite; and in either case there is a larger percentage broken in transportation than most engineers suppose. When there are different sizes of coal in the same car the smaller fills up between the larger, and a less quantity is crushed. This fine coal, if bituminous, can be readily used at the end of the route; a large part of the anthracite dust is sold, but at a reduced price, and the difference added to that of good marketable coal. Probably the form of cars could be changed so as to lessen this item of waste.

The loss in volatile matter—and even in fixed carbon—which coal, and particularly bituminous coal, undergoes, when allowed to heat in heaps exposed to the weather, amounts sometimes to 10, and possibly even 20, or more per cent. Coal should be kept dry, cool and under cover—an important matter well worth the consideration of railroad managers.

These are the principal items of loss. This Society of Civil Engineers, perhaps, is not so much interested in the mining of coal as its transportation; still, whatever affects the cost in the market of this prime commodity should receive attention from railroad, scarcely less than from mining engineers. The question of the waste of coal is being examined by a committee of our sister society—the American Institute of Mining Engineers, of which I have the honor to be a member; the committee has been more than a year engaged in collecting information, and, without doubt, the result of the investigation will have an important bearing upon the subject of mining and preparing coal.

Mr. McAlpine—I stated that three-fourths of all the freight from the West to the tide-water went by two canals—chiefly by the Erie; that left but two million tons to be carried by the four trunk railroads altogether. Reports referred to in my address before the Chamber of Commerce, show how much was actually transported both ways; although the quantity of freight is large, during the year these roads together moved eastward only two million tons of through freight, which is the class I alluded to.

I would mention that thirty years ago I ascertained the tonnage from the West to tide-water, by all the railways and water lines, to be nearly four times as much as that from tide-water to the West, while the values of the two were nearly equal.

Mr. Shinn—The statements I made of the cost of transportation were not of that upon a model railroad, but based on the work actually done by existing railways during the past year; they were made after full deliberation, and result from due investigation. I am prepared to prove them, and when I can possibly find the time, I will do so in detail.

LXIII.

BACK-WATER IN STREAMS AS PRODUCED BY DAMS.

A Paper by Prof. De Volson Wood, Member of the Society,

Read at the Fifth Annual Convention in Louisville, Ky., May 21st and 22d, 1873.

Charles Hermany, C. E., in the Chair.

One of the motives in presenting this paper is to profit by the discussion on it, which it is hoped will follow. Although the subject has caused much litigation, most of the information given in the courts is useless for scientific purposes. In my practice, I have had cases of back-water to investigate; in a particular instance, where I was called before a court, on one side it was testified by some, that according to observation, the effect of the dam in causing back-water, did not extend back more than one-third of a mile, and by others, that it was not visible for more than 200 or 300 feet; on the other side, parties testified, that according to circumstances by which they could not possibly be deceived, the effect was visible for at least $1\frac{1}{2}$ miles; and what was especially remarkable in the latter was, that the supposed rise due to back-water at this point ($1\frac{1}{2}$ miles back) equalled or exceeded the actual fall at the dam!

It need not be said to this Society that the problem admits of a theoretical discussion, and has received the attention of such scholars as Prony, D'Aubuisson, Weisbach, Rankine and many others, and they all show, upon theoretical grounds, that the effect of back-water, as caused by a dam in a flowing stream extends, or in most cases may extend, far

back of the hydrostatic level. And yet many men, called engineers, are not only ignorant of this fact, but deny its truth.

As this is a broad statement, I will cite examples in support of it. On a certain stream in New York are located several mill dams. One party complained that the dam next below his, caused the water to flow back upon his wheel and damaged him. An engineer, supposed to be the most scientific man in that section of the country, investigated the case. He ran a line of levels from the crest of the lower dam up stream to where he found a perceptible fall, and suspended work. He then reported that the dam did not cause the water to rise above that point; for, said he, in explaining the case, "water will stand at a level;" an argument which seemed evident to his auditors and employers. It would have been more apt to have said, "water never runs at a level." It hardly need be mentioned that had his levels been carefully taken, he would have found a fall in that part of the stream where the water appeared to be still. Another engineer who had settled several disputes involving damage from back-water, replied to the enquiry, how much allowance he made for the "piling up of water," as it is popularly called, that he never made any; he had heard of such a thing, but did not suppose that its effect was perceptible; still another strongly denied its existence.

These instances show great ignorance on the subject among engineers of a certain class; yet if they should turn to us for instruction, we might teach them certain general principles, and then would be forced to admit that we were unable accurately to determine what would take place in any given case. Before any physical problem can be investigated analytically, the laws of action which are involved must be known. With fluids, these are often so complex and so imperfectly understood that even in the more simple cases only approximate results can be obtained.

In the problem considered the laws of action are hardly continuous. The theory of permanent flow, which is the most general of any here applicable, assumes that there is no sudden change in the bed of the stream or in the direction of its current—which is contrary to the facts in almost every practical case. Besides this, the bed of the stream may frequently change from mud to coarse gravel or stones, trees and flood-wood may cause a partial dam, and there may be eddies, which seem to almost suspend the velocity and cause it to start anew, while in the immediate vicinity there may be rapids. By no theory can the quantity of flow in such complicated cases be exactly determined.

The remous proper seems to be an exception to the case of ordinary

flow ; for while the water at the surface is nearly still, the velocity near the bed in many cases is comparatively rapid, like a stream in a body of water—as the gulf stream, for instance, in the ocean. I am not aware that the analysis of such a case has been attempted. The problem is still further complicated by whirls at the dam and reversals of current, it in many cases being up stream near the shore, and down stream elsewhere.

Such are some of the peculiarities of this problem, which appears too complicated for ordinary analysis. D'Aubuisson, in his "Hydraulics," states that he formerly thought the theory of permanent flow would furnish a solution, but further consideration convinced him that it did not apply to these cases. He knew of no more reliable mode of computation than by St. Guilthem's empyrical formula. I believe we have no theory or formula which will determine with reasonable accuracy the amount of back-water beyond where the hydrostatic level intersects the natural surface of the stream.

More facts are required before reliable theories or formulas can be established. American literature on this subject is very meagre. I have failed to find the record of any case in this country which has been thoroughly investigated, if such exists. Engineers are obliged to rely for positive knowledge on a few cases which have been reported in Europe. Doubtless the engineering profession has valuable information bearing on this problem, which should be preserved. It is to be regretted that in this country, so much useful matter in the way of practical experience is lost, because it is not put on permanent record ; and I trust that this Society may not only so conduct its affairs as to be a means of intercommunication between its members, but also to induce them to report, for publication, all important matter which comes within their line of practice.

The few cases which have come within my knowledge, including two taken from the "Annales des Ponts et Chaussées," indicate that the effect of back-water, is considerable for some distance back of the point where a horizontal line, called the hydrostatic level, cuts the natural surface of the stream, and is—say from about one-third to one-half the latter. This is shown by the surveys of the rivers Wesser and Werra.

The river Wesser is about 354 feet wide, 2.46 feet deep, with an average fall for about 10 miles, of 2.33 feet per mile, quite uniform for the whole distance. It is especially interesting because it was accurately surveyed before and after the dam was erected, although, unfortunately for the purposes of accurate comparison, I was not able to find the record of the

survey of natural surface. But the profile of the remous is accurately constructed from the original record, which was found in the Astor Library. The straight line A B represents the average slope, and is evidently ideal, since the bed of the stream is so irregular as to cause the surface to be irregular also. D'Aubuisson, to whom this case was referred, reports that at the highest point observed, 4.33 miles above the dam, the effect of back-water was barely perceptible; hence we must infer that the natural surface of the stream was several inches above the average grade; and nearly, if not quite, coincident with the surface of the remous, as given in the drawing; and hence the natural surface, which is represented at A, was nearly if not quite coincident with C. It will be observed that where the hydrostatic level cuts the natural surface, which is 3.07 miles above the dam, the back-water causes an elevation of 15.2 inches by actual measurement, and hence is independent of any theory. This exceeds the effect, as given by any of the formulas used in such cases. I would call attention to this fact: the drawing indicates that the surface has a wave-like form—the waves being shorter at the upper end, and very long nearer the dam.

The difficult part of the problem is that which refers to the surface, above where it is intersected by the hydrostatic level. It is found that below this point, the contour of the bed has little or no effect upon the surface of the remous, so that a formula which will give the result for this in one case, will be nearly correct in similar cases. Above the hydrostatic level, the surface is very irregular, which is due in a measure to the contour of the bed. It is said in the report that the back-water practically disappeared at a certain point—which point cannot be determined from the average slope. The distance from that point, as reported, to where the hydrostatic level intersects the actual surface is nearly one-third of the whole distance observed, or about one-half the length of the hydrostatic level. If the actual and average slopes had been the same, there can hardly be a doubt that the effect would have extended above this point. It is quite probable that a casual observation of the velocity of the stream on the upper part of the remous would have led one to infer that the remous did not extend so far up stream; but whether so or not, it is quite evident that the extent of a remous cannot be determined with any degree of accuracy by simple observations upon the velocity.

The Werra—one of the tributaries of the Wesser—is a much more rapid stream. D'Aubuisson gives data* in regard to it, from which the

* Fall, 4.6 metres in 694 metres; quantity of flow, 8.5 cubic metres per second; depth of chute over dam, 0.344 metres.

following approximate quantities were computed: width of stream 80 feet, mean depth 1.7 feet, and fall per mile 3.88 feet. It will be seen that the effect of the remous on the part back of the hydrostatic level is more regular than in the Wesser—it vanishes quite uniformly.

The Kalamazoo creek is a stream similar to the Wesser, but with less fall; it is about 250 feet wide, 2.8 feet deep, and the fall per mile was about 16 inches. I had to examine it, and ascertain how far the back water extended. I took a series of levels, marked them on the drawing, and indicated the effect of the remous. In doing this I was guided in a general way by the theory that the remous will extend further in a sluggish than a rapid stream. The dam raised the water only 4.5 inches above the natural surface, and yet the remous probably extended three-fourths of a mile. It was claimed by the prosecution that the dam was higher in former years, and then damaged land more than a mile up stream.

The last case cited is the Pau Pau River, Mich. The width varies from 250 to 350 feet, the depth averages about 3 feet, and the fall was about 2 feet per mile for 5 miles. When I surveyed it, a dam erected several years previously, which caused back-water for about $4\frac{1}{2}$ miles up stream, had broken away, so that I could determine with considerable accuracy the normal condition of the stream. In showing the back-water on the drawing, I was guided partly by theory and partly by information gained upon the ground. The representation of the remous in this case is not the result of an accurate survey.

Mr. McAlpine—I have been several times in court, to testify upon these precise cases. Among the books and papers which I sent to the Society is, I think, my manuscript relating to an interesting case at Rochester, N. Y. I once had the Tennessee river surveyed carefully for six or seven months, from extreme low water to extreme high water. I will, at some future day, submit to the Society what I prepared twenty years ago on this subject, with the facts and measurements.

Mr. C. Shaler Smith—On the Mississippi, above St. Louis, and a short distance below the mouth of the Missouri, the effects of back-water may be observed at times on a very large scale. An ice gorge generally forms during the winter; within an hour after it occurs, the effect is apparent at St. Charles, in the slackened speed of the ice, and this is clearly perceptible for some time before any rise of the water can be detected. On one occasion at least, the speed of the water as well as of the ice must have diminished, as the floes were so heavy that their draught was fully five-eighths of the mean depth of the river, and any difference in speed would of itself have caused a rise of the general level.

Mr. Collingwood—I have measured a stream before and after the dam was erected, or rather taken levels after the dam had been removed, and conclude that the height of the back flow is rarely a large quantity. For instance, in one stream about 50 feet wide, where I made the survey, and then had the dam put in, the back flow was about 8 inches. It was the same in another case where the stream was from 50 to 100 feet wide; this stream was very sluggish, and at a point where it was 2 feet above the hydrostatic level of the dam, the rise was 5 inches, and consequently extended a long distance back. In a third case, of a small stream about 10 feet wide, 200 feet from the dam the rise was 2 inches, and at 500 feet one inch.

Prof. Greene—In a case under my charge, where the levels were taken before the dam was erected, and again afterwards, the hydrostatic level intersected the natural surface of the stream at 2,600 feet from the dam, and the effect of the back-water practically ceased at 3,900 feet. The stream was 70 to 80 feet wide and about 2 feet deep. This substantially agrees with the authorities on the subject, and Prof. Wood's conclusions.

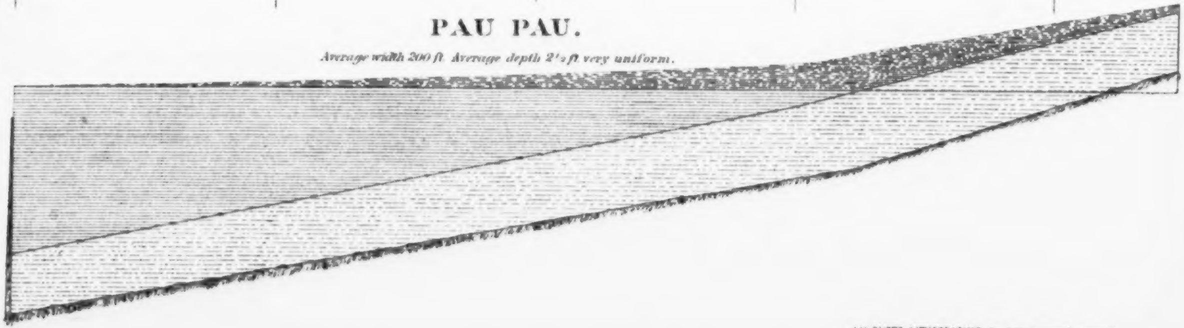
Mr. Chesbrough—One of the members of this Society, Mr. James B. Francis, perhaps as well known as any other, has written a work on hydraulics, in which there is much information in regard to this matter. I had an opportunity once to remove part of a dam; my associate was a man who hooted at the idea of water piling up. In this case, I concluded that the effect of the back-water did extend beyond the intersection of a level line through the top of the dam with the bottom of the center of the stream. A suit was the immediate cause of the investigation which I was employed by one party to make, who did not call me to testify, because my evidence would have been against him. The decision confirmed my opinion that the dam did affect the mill privilege. One object I had in this particular case was to ascertain what effect would be caused by raising the water below the dam; and until the surface was brought nearly to the crest, I failed to discover any.

Mr. Worthen—Mr. Francis made some delicate experiments on this point, which showed that raising the water below a dam produced no perceptible effect on the water above, until the surface below the dam was brought nearly level with the crest, when the surface above suddenly fell a short space.

Prof. Wood—I once arranged a box so that water would flow from it freely, the spouting vein having a fall of 16 inches on the outside. In three successive trials, the water surface fell a fixed space in times, as fol-

PAU PAU.

Average width 200 ft. Average depth 2½ ft. very uniform.



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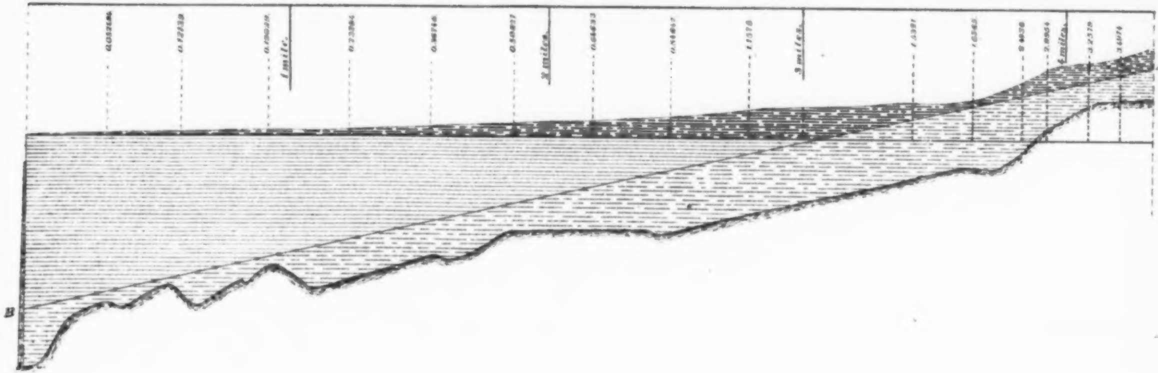
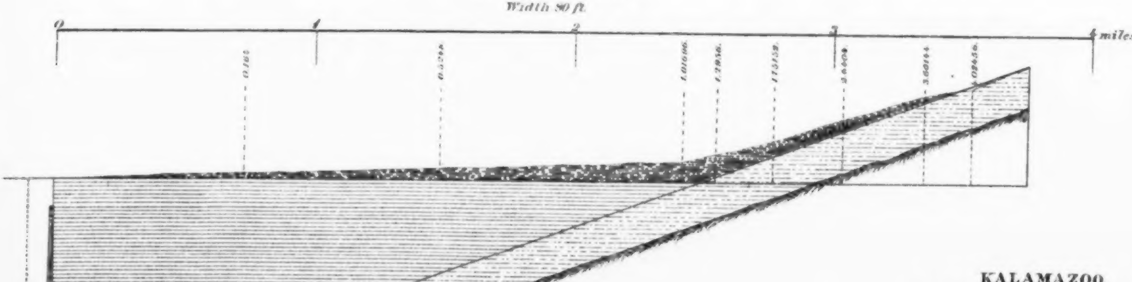
WESSER.

Width 353.54 feet.



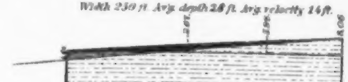
WERRA.

Width 80 ft.

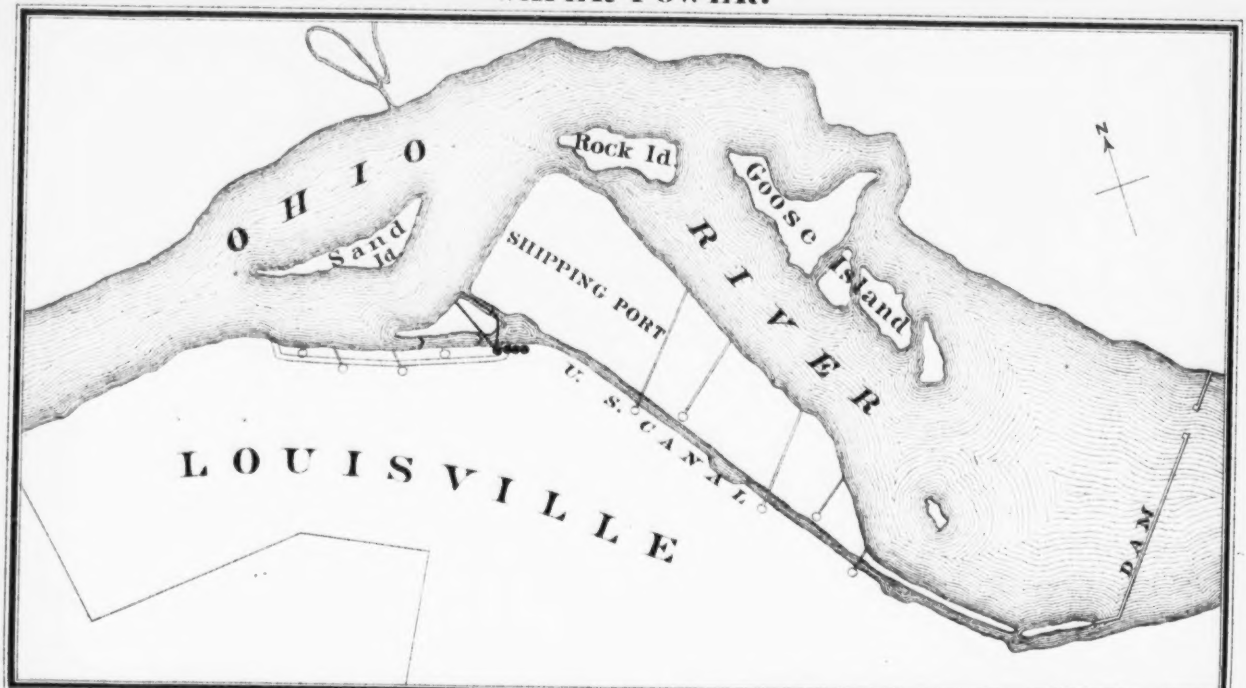


KALAMAZOO.

Width 250 ft. Avg. depth 2.8 ft. Avg. velocity 1.4 ft.



SKETCH
showing various **PLANS** *for obtaining*
WATER POWER.



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lows: 1 minute 17 seconds, 1 minute 18 seconds and 1 minute 19 seconds—the mean being 1 minute 18 seconds. I then placed a vessel of water so that the surface was about one inch below the centre of the discharge tube, and repeated the experiment; with the same quantity of flow as before, the times were: 1 minute 20 seconds, 1 minute 17 seconds, and 1 minute 19½ seconds—the mean being 1 minute 18⅚ seconds. This showed that raising the water at the exit, as in this trial, barely produced a perceptible effect. More delicate experiments might possibly give a slightly different result ; my object, however, was chiefly to see what result would be apparent with an ordinary apparatus.

I wish to enforce the idea stated in the beginning of this paper, that we ought, as a Society, to secure a record of practical results, such as have been brought out in this discussion. In this country, perhaps more than any other, much valuable experience is lost by not putting it in shape for preservation.

Note.—Members are requested to communicate to the Society any facts and figures possessed by them bearing upon the subject of the preceding paper.

LXIV.

WATER-POWER OF THE FALLS OF THE OHIO RIVER.

A Paper by Morris S. Belknap, C. E., Member of the Society,

Read at the Fifth Annual Convention, in Louisville, Ky., May 21st and 22d, 1873.

Charles Hermany, C. E., in the Chair.

The Falls of the Ohio river consist, as is generally known, of a series of rapids nearly opposite the city of Louisville, occupying the river for a length of about three miles ; a great obstacle to navigation, except when covered by high-water, they caused in former years most of the commerce on the river to come to a halt before passing them. The basin above the Falls being a stopping place, a town naturally sprang up, and thus they were the primary cause of the existence of the present city.

To relieve the navigation of the river from the trouble and danger arising from the descent of the Falls in the lower stages of water, private enterprise in the first place, and afterwards the general government, at a cost of about $4,000,000, have constructed and enlarged a canal of some two miles in length, by which boats of the largest size can be conveyed

around them without danger, and with but small loss of time. This canal here is denominated the United States canal, and it is presupposed that arrangements can be made with its owners for its use, at a fair remuneration, for water purposes.

From being long regarded as a great and serious obstacle to the commerce of the city, the Falls, as soon as it became apparent that Louisville was to be a manufacturing town, attracted considerable attention from those interested, as a possible source of power, and in late years much has been said and written of the millions of horse-power that were allowed daily to waste without application to any useful purpose. The interest thus manifested has in the past provoked several inquiries into the possibility of utilizing the vast force thus lost, but none have, it is thought, offered the most economical solution, and all have tacitly assumed a financial success, without seeking to answer the important question "Will it pay?"

It is not proposed in this paper to treat the subject exhaustively. Time and opportunity for the instrumental and other work necessary for a complete examination has been wanting, and therefore the estimate and figures given are but approximate—sufficiently near the truth, perhaps, for the attempt here made to set forth some of the various solutions apparent and proposed, present a special one, draw attention to a few points that may have been overlooked, and give direction to future research.

Of the origin of these rapids there is a perfect explanation in the geological record of the bed and shores of the stream, which give ample testimony on this point. Indeed, from the extreme richness of the fossiliferous bed exposed at low-water, the general study of the science has been much advanced. According to the conclusion of Drs. Yandell and Shumard, who made many years ago the most complete report yet published on the geology of the Falls, the present location of the rapids was entirely covered by a blue clay shale, through which the current had its passage very much on the Louisville side; in fact, the very site of the city, and as far back as the hills, must once have been covered by water, as the strata—the shale—overlying the harder rocks, has in the entire city basin disappeared, and there is a vast deposit of sand instead. With small powers of resistance to the combined action of air and water, the shale was soon disintegrated and washed away, and the more permanent rocky bed exposed, with a strong dip to the south and west—a dip of more than twenty feet in two miles. The ledges worn in their less resistent places form the rapids, which, from being as stated one of the primary causes of the location of the city, have long been looked to as one of the

greatest agencies in its future development and wealth. As of geological interest, it may be stated, that the ledges thus forming the rapids consist, in the order of their superposition, of—

1. Upper limestone—sub-crystalline, 8 ; water limestone, 12 = 20.

2. Shell limestone—sub-crystalline, characteristic fossils, tribolites, etc., 16.

3. Coraline limestone—upper coral, 20 ; lower coral (by estimate), 20 = 40.

It is in the upper one of this series that occurs the argeliferous limestone, from which the fine cements so well known to the engineering and building interests are made, an important manufacture in the industries of Louisville.

As the value of any waterfall for industrial purposes depends materially on the regimen of the stream on which it occurs, it seems proper first to inquire into the influences which affect it in this instance. The inquiry may, in this case, be brief, from the fact that, *a priori*, it is evident that the volume of water in the Ohio is ample at any stage to supply the probable demands of industry for many years to come. The Ohio river above the Falls is fed principally by the two streams, the Allegheny and Monongahela. The area of its drainage basin, above Wheeling, including those on the two streams just mentioned, is estimated at 21,337 square miles (Ellet), with an average flow at that place of about 1,560,000 gallons per minute. Observations at Louisville place this daily average flow for the same stage, at about 3,480,000 gallons per minute.

But the average discharge is not all that is needed for this purpose. It is the daily discharge that is required, the period of its maximum and minimum, and what effect the variations have on the total fall. Fortunately, the authorities in charge of the United States canal, and also of the city water-works, have the record for many years back of the variations in the height of water above the Falls, and from them has been obtained the tables and data upon which these calculations are based. The erection of the wing dams shown on the map accompanying this paper has had the effect of forming a basin and increasing the depth of the entrance of the canal ; when completed, it is estimated that it will increase the difference of level between the upper and lower basins of the Falls about 3 feet, adding very materially to the power it is proposed to use.

By a comparison of the heights, as observed simultaneously above and below the Falls, it appears in generalizing the results, that at the period

of low-water, when, as has been stated, the daily discharge is 3,480,000 gallons, the total fall is about 26.67 feet, with one foot above extreme low-water on the gauge mark about the Falls. As the river rises, this difference disappears rapidly, until at a stage of 12 feet by the mark about nearly all fall is lost, and the surface of the water is at its normal slope. In other words, the rise in the lower basin is to that in the upper as three to one.

Since at high-water all fall is lost, we must inquire at what time, and for what length of time this occurs. The record just mentioned gives, as the average of year's observations, that:

For 3 months and 17½ days there are 2 feet of water or less:

For 6 months and 3½ days there are 2 to 7 feet of water:

For 1 month and 18 days there are 7 to 12 feet of water; and

For 21 days there are over 12 feet of water.

This would correspond to differences of level for these various stages of

26 feet for 3 months 17½ days:

13 " for 6 months 3½ days:

6 " for 1 month 18 days, and

Nothing for 21 days.

It may, therefore, be safely assumed that for nine months the fall is considerable, being an average of nearly 21 feet, at the same time that water is sufficiently plentiful for all purposes; that the time during which the power is absolutely lost is about from three to four weeks, and that this occurs generally in the winter season.

That most industrial manufacturing establishments can, without loss, cease operations for that length of time is not very probable, but in point of fact most of them do stop for fully that length of time, at some period in the year, for repairs, refitting, etc. Notwithstanding, to be within all possible chance of error, it will be assumed, in this comparison, that the water power practically fails for three months in the year. Knowing, therefore, that there is an abundance of water, the amount of its fall, and the period during which it can be used, these questions may be examined understandingly:

First, whether it is physically practicable to obtain power from it; if so, what is the best method of so doing?

Second, will it be profitable to use such power after it is obtained?

Taking up the first question, a glance at the topography of the country from the head of the Falls to the foot, at Portland, enables one to say at once there is no insurmountable or even troublesome obstacle in the

way of erecting the necessary works. What the nature of these works should be, leads at once into the discussion of the various plans proposed for the utilization of this power, all of which are based on the use of the more improved class of hydraulic machinery. Current wheels, etc., while they may render occasional service, are essentially crude in their design and limited in their application. It is evident, therefore, that in effect there must be a supply canal or race, of the level of the upper basin, and a discharge-pipe at the level of the lower one. This can be accomplished in a variety of ways, each more or less advantageous or expensive.

1st. By means of a supply canal starting at and from the level of the upper basin, and running to a certain extent parallel to and resembling the United States canal, the mills situated along it to draw their supply from the canal, and after passing it through the proper hydraulic machinery to discharge it, either into a collecting pipe or sewer running parallel to the supply canal, extending nearly its whole length, and emptying into the river at some point below the lower locks of the United States canal, or directly into the Ohio river at a point as nearly opposite as may be to the position of the mill.

2d. To use the United States canal as a supply basin, and discharge in either of the methods first mentioned.

3d. To construct a canal leading from the extremity of the present United States canal, forming a continuation of this last, running through a part of Portland, and discharging the waste water directly from each mill into the Ohio river.

4th. To erect the necessary turbines or other hydraulic machinery at the lower end of the United States canal, drawing the water supply from it, and discharging the waste into the river at the nearest point—the power thus obtained to be distributed over the building area within available range, by means of the wire rope transmission system.

The first plan proposed, while not physically impossible, is almost impracticable, by reason of the enormous expense involved in carrying it out. The canal would have to be built at the same level as the United States canal, and the cost of that work up to the present time, nearly $4,000,000 (and it is not yet finished), warns private enterprise against a similar undertaking. Besides, if permission can be obtained to take the United States canal, such work would be unnecessary. A waste-pipe leading to the lower locks—and this is the method of discharge, which would give the maximum of power—would likewise, of necessity, be large, and be excavated in the solid rock at a great depth, at a cost, if

anything, more than that of the supply canal. The system of discharge by means of pipes directly into the river, offers but slight advantage in point of cost, if any, when the combined length of the several waste-pipes are considered, and the fact that each one from the southern side would have to pass under the United States canal and across Shippingport. There would be sacrificed, also, by this system, a large portion of the available power, as only the lower mills would have the benefit of the entire fall, the extreme upper ones having very little—the water on the Kentucky side being considerably higher than in the Indiana channel. While, therefore, it is very desirable on account of the easy access to railroads and the adaptability to manufacturing purposes, of the ground lying along the line of the proposed canal, that it should be occupied, neither of the first solutions admit of this, except at an insurmountable cost.

The second plan proposed—to use the United States canal as a supply basin and discharge, as mentioned before—saves at once of the cost of the canal; but the cost of the discharge-pipes running to the lower locks remains as an obstacle to the adoption of the first method of discharge in this system; and for the second method there is still the small fall obtained for the money expended. It may be that this solution can be advantageously used in some special instances, but for general purposes it is believed a more available and economical way may be found.

The third plan proposed—one put forth some years ago by Mr. Miller—is to form a dock or basin in the United States canal at or near the lower locks; to start a canal from this basin, running through a part of Portland, and have the mills situated on either side of this canal empty directly into the river. This canal being thus entirely below the Falls, the mills along its course could realize the benefit of the total fall. Again, the rock having disappeared from the surface, in fact, nearly from the ground at the depth required, the excavation would be mostly in earth, which would materially reduce the cost. On the other hand, Portland is far removed, not only from the business centre, but also from railroad transportation, the general source of supply and export. The canal and discharge-pipes would also necessarily be costly compared to the amount of power that could be used; therefore, the results of this project are not so available as of some of the others.

The last plan proposed—the one which appears, after much reflection, to be not only the most feasible, but the most economical and the easiest put in operation—is to use the United States canal as a supply basin,

locate the necessary machinery at its lower end, and discharge the waste water into the river at that point by the shortest possible route—the power thus obtained to be distributed by means of wire-rope transmission over the neighboring territory. This system would cost the least, as but a short discharge-pipe would be required, and that at a point where there is but little rock; the maximum effect would be here obtained, for the total fall could be used at all times. The machinery, therefore, would be less expensive than where only a part of the fall could be utilized, and the consumption of water would be less. Again, factories could be located at any point within reach, on high or low ground, and power given them without difficulty. There would be less trouble in measuring the amount employed, as this could be done by mechanical means, while the quantity of water used could be ascertained much more easily than where it is divided between a number of establishments. The distribution by wire-rope transmission is not only practicable, but the simplest and cheapest method that can be devised, and the loss of power by such transmission in the limited range in which would probably be situated the establishments dependent on the central source of power, would be small—in all probability not exceeding an average of five per cent. This appears to be the most economical and practical way of utilizing the power of the Falls of the Ohio. It may not be the most general solution, and, it is possible, may be inadequate to the demands of the future, but that future seems too far distant to warrant a provision for it in the shape of extensive and costly hydraulic works.

Accepting, then, this last as the most economical of the various plans, the second query naturally arises: "Is it desirable? Will it pay?" This is the financial side of the question, and the important one; for if nothing is to be gained by the substitution of water for steam-power it matters little whether the change be practicable or not.

The comparison necessary to determine this question may be made upon two hypotheses:

1st. That the establishments using water-power are of such a nature as to be able to stop whenever the power fails—that is, to vary their running according to the fluctuations of the river.

2d. That they must have a continued power, and, therefore, supplement the water by steam, either as an aid when, by reason of the head diminishing, the water-power is not sufficient, or to replace it when it fails altogether, as in periods of high water. This is the only true and rational basis upon which to proceed, as it is evident that no manufacturing

establishment could stand the perturbations that would be inevitably introduced into its business if dependent on the uncertain fluctuations of a stream. The question, then, in its simplest form, is: "Whether such a combination of water and steam is more economical than steam alone?"

The cost of the two systems may be compared, and reductions made from them. One thousand horse-power may be assumed as being ample to satisfy the demands of industry for many years; besides, the results would be proportionately true for a greater power; consider this as produced by ten engines of one hundred horse-power each—that is divided among ten establishments using steam, and as being produced by four or more turbine wheels; further, admit that the water-power is available for nine months in the year, and that for three months it is entirely replaced by steam; the cost of erecting the necessary hydraulic machinery for the realization of this power is estimated not to exceed $25,000, and with a fall of nineteen or twenty feet it would use about 35,000 gallons of water per minute, which—as the right has been reserved to tap the canal—would cost little or nothing. Suppose the labor about the turbines to be worth $12 per day, neglecting other expenses, such as wear and tear, repairs, etc., on both sides of the account, the annual expenditure for water-power would stand thus:

Labor for 225 days, at $12	$2,700
Interest on $25,000 for 3 months (time during which water-power machinery is supposed to be idle) at 10 per cent	625
Total	$3,325

The cost of a steam-engine of one hundred horse-power, of a desirable class of workmanship, with all appurtenances, cannot fall far short of $20,000. Ten such would, therefore, represent an investment of $200,000. Estimating the labor in attending on each engine, of such a class, at $10, and fuel as one hundred bushels per day at 12 cents, neglecting, as before mentioned, wear and tear, oil, repairs, etc., etc., and recollecting that the engine would be only in use three months in the year, the annual expenditure for steam power (under the combined system) would be about as follows:

Fuel for 10 engines for 75 days, at $12	$9,000 00
Attendance on 10 engines for 75 days, at $10	7,500 00
Interest on steam machinery for 9 months, say on $200,000 at 10 per cent. for 9 months	15,000 00
Cost of 1,000 H. P. for 3 months	$31,500 00

Cost of 1,000 H. P. for 9 months........................ $3,325 00

Total cost of 1,000 H. P. for 12 months.......... $34,825 00

If steam were relied on alone there would be no time at which the capital invested would be lying idle, and therefore the item of interest could be left out of the account. In such a case the estimate would be about as follows:

Fuel for 10 engines per year	$36,000
Attendance 10 engines per year	30,000
Oil and waste, etc	2,000
	$68,000

Or about double what the same power would cost under the combined system. It is therefore tolerably certain that if the water-power is made available on some cheap plan, there is a pecuniary advantage in employing it even by those who are compelled to have a steady power, and supplement the water by steam. If there is any advantage to these, there is much greater to those who can use an intermittent power. It may be remarked that no compensation for the water used has been allowed. It is not known what arrangements were made respecting this when the canal was turned over to the United States authorities, but a low price per thousand gallons would make the two systems equal.

The subject is one of great importance, and has been but outlined, with a hope to excite inquiry and study concerning it, and that hereafter the commerce of the country may reap much benefit from it.

Mr. McAlpine—I have had occasion to look into the movement of the immense products of this western country to the sea-board, and in a report upon the subject, I called attention to the remarkable fact that almost no water-power exists on the western side of the Alleghany mountains. The products of this particular section of country are for the most part cereal, and difficult to transport over long lines of railway; hence it is necessary to condense them into pork, beef, whiskey and the like, which will bear long transportation with economy. Water-power, such as that at Louisville, is of immense importance in connection with this matter, and I have no doubt it will be used. I have been informed that at Lowell, Mass., water-power costs about one-third as much as steam-power.

In speaking of the manner of discharging the waste water, a sewer has been mentioned; this must be substantially as large as the canal of

supply, otherwise the head would be destroyed; a direct discharge into the river would probably be best.

In discussing the methods of transmitting power, the use of compressed air as an agent deserves attention. The late Gen. Rodman considered this plan, in reference to the erection of works at Rock Island. President Barnard, of Columbia College, in his report as one of the Commissioners to the Paris Exposition, gives a valuable formula in connection with the use of compressed air, and cites a number of experiments which were made in France. Mr. Charles S. Stoddard, who was sent out to examine the Mont Cenis tunnel, remarks that where air was sent through an eight-inch pipe three miles in length, it was impossible to measure the loss with any instrument that they had.

Mr. Collingwood—The subject of transmitting power by compressed air should be carefully examined. I am satisfied there is a considerable loss due to loss of temperature. Much of the heat developed by compression is lost during transmission, and a loss of tension ensues. I have in mind a case in which an important project was discussed wherein it was assumed, apparently without question, that all of the power used to compress the air could be transmitted several miles and made available.

Mr. J. Dutton Steele—I have had some experience in the transmission of power by compressed air; in reference to the loss due to a loss of heat, mentioned by Mr. Collingwood, I think there is little in it, practically; there is little, if any, loss in the transmission by changes of temperature to which the air may be subjected in its passage from the compressor to the engine. Although there is an increase in temperature of the air, due to compression, which heats the air pumps so that it is necessary to use water to keep them cool, this does not affect the general pressure in the conduit pipes, or the engines; indeed, it is surprising how soon the temperature in the pipes is reduced to that of the surrounding atmosphere.

Careful observations show the loss of power as transmitted by compressed air in tunnel-work, that is the difference in pressures of the steam used to drive the compressors, and of the air at the engines, to be about equal to that due to the friction of the machinery employed.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

XLV.

IRON HULLS FOR WESTERN RIVER STEAMBOATS.

A Paper by THEODORE ALLEN, M. E., Member of the Society.

READ AT THE FIFTH ANNUAL CONVENTION, IN LOUISVILLE, KY., MAY 21ST AND 22D, 1873.

CHARLES HERMANY, C. E., IN THE CHAIR.

Owing to the increasing scarcity of ship timber, and the consequent greater cost of wooden vessels, attention has been more and more turned to the substitution of iron for wood in the structure of hulls for vessels of all descriptions, with steam and sail, for ocean traffic or river service. Abroad, in England especially, iron has almost entirely superseded wood for this purpose, and in this country at the present time out of twenty-one ocean-going steamers in process of construction but four are being built of wood. The greater durability, the lesser weight, the small cost for annual repairs of iron hulls, when compared with wooden hulls, has, even on this side of the Atlantic ocean, overcome the slight increase of their first cost, and the wooden ocean steamer is almost a relic of the past. On the lakes the increase in the number of iron vessels built each year for the past ten years, shows that even in that section of the country where wood should still be cheap, the iron hull is rapidly winning its way to public favor. The use of iron hulls for Western river boats has excited considerable discussion during the past few years, and it is for the purpose of presenting the advantages of the iron hull in more of a professional form than heretofore, that this article has been

prepared. To introduce the subject in such a manner as to fairly compare the two methods of construction, it is proposed to give a history of the use and disposition of the material in iron hulls for light draught purposes, up to the latest examples published ; to show the various devices which have been used to obtain the great strength, with lightness, necessary for vessels of this class, and to present a plan which, based upon past experience, shall be specially adapted to the requirements of steam vessels designed for service upon the shallow waters of our Western rivers.

The first iron canal-boat of which any record has been preserved, (Grantham's "Iron Ship Building") was built in 1787 ; she was 70 feet long, 6 feet $8\frac{1}{2}$ inches beam and weighed 8 tons. Some of these early vessels were broken up after 40 years' service ; the first iron steamship, the "Aaron Manby," built in 1822, was the first also which went to sea. She was designed for the navigation of the Seine, and carried a cargo direct from London to Paris. Iron river vessels were early built for the Thames, and were soon after introduced in India, upon the river Ganges. On the latter river they were, until 1844, small boats used mainly as tugs, and belonged to the government of India. At that time the Ganges Steam Navigation Company was formed, and from designs prepared by A. Robinson ("Steam on the Ganges") there were five vessels built. They were first put together in England, then taken apart, shipped to Calcutta and there re-erected and riveted up. A short description taken from Robinson's work will give an idea of the progress of iron river constructions up to that time.

The first of these boats, called the "Patna," was launched in 1846. She was intended to carry both passengers and freight, and in model and plan of deck and saloons, was quite similar to the Mississippi steamers. Her dimensions were :

Length on load water-line	195	feet.
Beam over hull	28	"
Beam over paddles	$46\frac{9}{12}$	"
Depth of iron hull	$7\frac{9}{12}$	"
Bottom of hull nearly flat.		
Displacement at 2 feet draft	205	tons.
" " 3 "	328	"
" " 4 "	455	"

The peculiarities of construction were as follows : "There is no external keel; it is replaced by an internal one or keelson, formed of a light, hol-

low iron beam, 2 feet deep and 9 inches wide, which is riveted to the inner frames of the bottom of the floor. Between this keelson and the iron deck beams, and riveted at their upper and lower sides to both, are light, stiff stanchions of iron, which have the effect of both trusses and ties; binding the floor and deck together. The sides of the vessel are vertical, and the iron frames which run up to form them, finish at the gunwale in a strong cornice, formed of angle iron and a narrow plate. The heads of the frames, the upper edge of the top strake of plate, and the ends of iron deck beams, are thus all riveted together. The powerful connection by this means formed between the bottom and floor and the midship trussing, constitute the entire hull into one large, hollow beam. The sides themselves are for a third of the vessel's length amidships strengthened by diagonal ties, crossing the ribs or frames at an angle of 45 degrees, and riveted to each rib. All the iron in the frame, flooring and shell is of light scantling, but of a quality and make giving the greatest tenacity and strength." "The paddle-boxes are built upon the sides of two light, hollow beams, which cross the vessel under the deck, and project beyond the sides for the purpose; the paddle boxes are framed of angle iron. The entire weight of the vessel, with paddle-boxes, but exclusive of machinery, cabins and stores, is 142 tons." The weight of engines, boilers, propelling machinery and engine bearers is 106 tons; cabins and upper deck 12 tons; water in boilers, coal, stores, &c., 46 tons; making a total weight, with steam up, of 306 tons, on a light draught of 2 feet 10 inches. The speed, by log on the trial, was 11½ miles per hour.

The last two of the five vessels were of greater dimensions, and having been built after the trial of the first vessels, may be taken to represent the improvements suggested by the running of the previous boats. Their general dimensions were:

Length on load water-line	250 feet.
Beam on load water-line	38 "
Beam over paddles	66 "
Depth amidships	10 "
Weight of vessel alone, including paddle-boxes and deck houses, but without machinery	224 tons.
Engines, boilers and wheels	134 "
Light draught, without water in boilers	$2\frac{2}{12}$ feet.

"The chief difference between these vessels and the 'Patna' consists in the deck being convex or curved upward transversely, like the back of

a violin, and in the bracing or trussing between the deck and the floor of the vessel being in the form of a diagonal lattice-work instead of vertical bars or stanchions, as in the former vessel." "The curvature of the deck was admissible from the absence of cabins and the diagonal framing or spine, from the circumstance of the engines being non-condensing and entirely above deck." As one of these vessels, the "Mirzapore," is considered an excellent example of transverse framing, having great strength for her weight, she is illustrated by a longitudinal section.

All vessels up to this time had been constructed in the same general manner as had been previously followed in the construction of wooden hulls, iron frames being substituted for the wooden ribs, and iron sheathing for the wooden planking. At about this time, however, attention was turned to the strength and stiffness of plate-iron structures. Fairbairn took out his patent for a cellular beam; Stephenson projected the Britania Bridge, and to test the strength of rolled beams, riveted joints, hollow beams and plate-iron work of similar character, and to determine the relative strength of such constructions, a very thorough series of experiments was tried under the direction of William Fairbairn. The deductions from these experiments developed certain formulas and constants which may be said to have since governed the profession in their calculations upon all structures of wrought-iron.

J. Scott Russell, then engaged in iron ship-building on the Thames, seems about this time to have given attention to the question whether in following out the same method of structure in the iron as in the wooden hull, ship-builders were deriving the best results from the material used. Calculations proved that the hulls of even the best vessels, as then constructed, possessed an excess of strength when considered with reference to the pressure exerted by the water against the sides and bottom, but were deficient in longitudinal strength to resist rupture when supported only at the ends or in the middle; side keelsons and deck stringers were used to obtain strength, but were expensive to construct when intercostal, and occupied valuable room if run upon the top of the floors. In order to obtain this longitudinal strength to a greater degree, with less material, Russell introduced what is known as the "longitudinal system." In this system, as the name implies, the frames, in place of being ribs and running athwartships or transversely, run from bow to stern, are kept rigid by bulkheads and intercostal floors or "partial bulkheads," as he terms them. Increased strength at the bow to resist compression or collapse, in case of collision, and greater stiffness at the stern

are also incidental advantages of this system, especially valuable for river vessels. E. J. Reed, late chief constructor of the British navy, says: "It has been objected to the longitudinal system of framing, that a greater space of unsupported bottom plating is left between the frames than is the case in the vertical (transverse) system. But it has been stated in reply that in case a vessel with transverse frames strikes on a rock, those transverse frames become immediately the most certain agents of destruction to the bottom of the ship, while in the longitudinal system the weakness existing is precisely what is wanted, for it allows the plates between the longitudinals to be indented or even torn through without the general structure of the ship becoming injured." ("Ship-building in Iron and Steel," pg. 95.)

The "Annette" was a vessel especially constructed to show the comparative strength of the two systems with equal weights of material. Russell (in his "Ship-building," pg. 371) gave for the "Annette" longitudinal system an increase in stiffness against sagging or hogging of 5 to 4, or a gain of 25 per cent.

This system of construction has been generally adopted for light draught vessels in England. Russell gives the following description of one built by him (Russell's "Ship-building," pg. 618). "The vessel is about 200 feet long and only 6 feet deep, or about 33 times as long as deep, and to get strength without weight in such a proportion is extremely difficult. She was to carry engines capable of driving her at a speed of 11 to 13 miles on the Thames on 20 inches draught. The general arrangements by which this result was accomplished are as follows: All internal frames were at once abandoned, and the whole ship was built of bulkheads longitudinal and tranverse, with longitudinal stringers between and partial bulkheads; in short, the whole structure might be said to be cellular; the difficulty was to get sufficient strength in the center of the ship to carry the weight of the engines and boilers, amounting with water and fuel to 150 tons, this difficulty arising from the extreme shallowness of the boat. I consequently decided that the walls of the cabin on deck should be added to the effectual depth of the ship, and I would consider her as a longitudinal girder, having that depth in the center, and construct her accordingly. I therefore made 105 feet of the middle of her length into a couple of plate girders, 15 feet deep, and these form the sides of the cabins, being perforated with doors wherever convenient; towards the two ends, these longitudinal girders are prolonged under the deck 35 feet at each end beyond the cabin girder, so

that effectively these deep girders give strength to the ship through her whole length." The general dimensions of this vessel were as follows :

Length on load water-line	198 feet 3 inches.
Breadth over hull	38 "
Breadth across paddle-boxes	60 "
Depth at side	6 "
Draught of water ; light	1 " 8 "
Draught of water ; laden	2 "
Indicated horse-power	688 H. P.
Area midship section	75 sq. feet.
Area load water-line	6,474 " "
Load, displacement (2,240 lbs.)	331 tons.
Displacement between 1 ft. 8 ins. & 2 ft.	$61\frac{2}{3}$ "
Grate surface	72 sq. feet.
Fire surface	3,300 " "
Weight of iron in hull	103 tons.
Weight of engine	$40\frac{6}{10}$ "
Weight of boilers	$37\frac{4}{10}$ "
Weight of water in boilers	22 "
Diameter of paddle-wheels	14 feet 4 inches.
Length of buckets	9 "
Revolutions of engines per minute	38
Pressure of steam	25 lbs.

Another steamer, intended for towing on the Rhine, also built by Russell, of about the same length, but having only 25 feet beam, constructed upon the longitudinal plan, had a load draught of 3 feet. She was 9 feet depth of hold, and her longitudinal floors, except under engines, were 16 inches deep only. The engines, boiler and water weighed 128 tons, her displacement at a load draught of 3 feet was 294 tons ; consequently the weight of hull complete, with the coal and the small cargo she may have carried, could not have exceeded 166 tons.

The information I have been able to obtain in regard to American practice is meagre ; so far as ascertained, the boats have been built exclusively on the transverse system, although longitudinal bulkheads and stiffening frames have been used. Haswell (pg. 650) gives the following particulars of a stern-wheel steamer :

Length on deck	110 feet.
Beam over hull	14 "
Beam over guards	22 "

TRANSVERSE SYSTEM

Length on Water Line	250 feet		Displacement at 5 feet ([illegible])		960 tons
Breadth of Beam	38 "		Weight of Vessel	"	224 "
Depth Amidships	10 "		Weight of Machinery	"	134 "

BUILT 1848

LONGITUDINAL SECTION

MIRZAPORE

SCALE 1"=16'

—

EXAMPLES OF

SHALLOW RIVER

STEAMERS.

LONGITUDINAL SYSTEM

Length on Water Line	190 ft 3 ins.	Displt at 2 ft. Load Draft ([illegible])	331 tons
Breadth of Beam	38 ft.	Weight of Iron in hull	103 "
Depth at Side	6 ft.	Weight of Machinery	78 "

LONGITUDINAL SECTION

J. SCOTT RUSSELL'S PLAN

SCALE 1 IN.=16 FT.

Depth of hold	3 feet 6 inches.
Load draught	$1\frac{1}{10}$ "
Two cylinders, diameter each	10 inches.
Stroke of pistons	3 feet.
Revolutions per minute	33

Plating of hull, keel No. 3 ; bilge No. 4 ; bottom No. 5 ; sides Nos. 6 and 7 ; frames $2\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$ inches, spaced 20 inches ; displacement 33 tons at load draught of $1\frac{1}{10}$ feet. Messrs. Harlan, Hollingsworth & Co., have constructed several light draught iron vessels, one a stern-wheel steamer, the "Vengochea," built in 1864, of dimensions as follows :

Length between perpendiculars	155 feet.
Beam over hull	26 "
Depth of hold	5 " 6 inches.
Two cylinders, diameter each	21 inches.
Stroke of pistons	6 feet.
Mean launching draught, with boilers in	$17\frac{3}{4}$ inches.

The Delaware Iron Ship-building Works constructed in 1869 a light-draught steamer called the "Novelty," dimensions as follows :

Length of deck	216 feet.
Beam over hull	24 "
Depth of hold	5 " 6 inches.
Mean draught, with machinery, boilers, water and fuel	2 " 1 inch.

The vessels above described will fairly show the present method of construction adopted for river steamers where iron is used. To, however, compare the iron hull with the wooden one, I have taken a wooden vessel of recent construction, combining in its structure and model the latest improvements which experience has shown to be best for the Western rivers ; and preserving the same lines and contour, and have replaced the wooden hull with one of iron.

Before entering into the comparison, I will enumerate the primary requirements for a light draught vessel. First—The skin or sheathing must be of as light a material as will withstand the pressure of the water, with the ability to resist the shocks caused by grounding or collision with snags. Second—The frame for stiffening the sheathing and preserving its form must be so disposed as to provide the greatest amount of longitudinal strength with the weight of material used. Third—The bow and

bilges must be designed with especial reference to the grounding of the vessel, and the dangers of river navigation due to obstructions.

To meet the first requirements, the strength necessary to resist what may be termed the punching of the bottom, only, will be calculated upon, as the pressure of the water is so slight at the draught sought, that it need not be taken into consideration. To test the relative strength of wood and iron for the skin of a vessel, William Fairbairn, in 1838, tried the following experiments (Fairbairn's "Iron Ship Building," pg. 79): "A plate was fastened upon a frame of cast-iron 1 foot square inside, and 1 foot 6 inches outside. The sides of the plate when hot were twisted around the frame and firmly bolted to it. The contraction by cooling caused it to be very tight, and the force to burst it was applied in the center. This was done in order that the force might in some degree resemble that from a stone or other body, with a blunt end pressing against the side or bottom of a vessel; a bolt of iron terminating in a hemisphere 3 inches in diameter, had thus its rounded end pressed perpendicularly to the plate in the middle." Four experiments were tried with the best Staffordshire iron, two with plates ¼-inch thick, and two with plates ½-inch thick. The results were the plates ¼-inch thick were burst with a mean force of 16,779 pounds; and the ½-inch plates with a mean force of 37,723 pounds. Fairbairn observes "here the strengths are as the depths (nearly), requiring double the weight to produce fracture of a ½-inch plate, as had previously burst the ¼-inch plate." The next experiments were made upon good English oak, of different thicknesses and of the same width as the iron plates; the specimens were laid upon solid planks 12 inches asunder, and by the same apparatus the rounded end of the 3-inch pin was forced through them as follows:

"Mean strength from planks 3 inches thick, 17,933 lbs.

"Mean strength from planks 1½ inches thick, 4,406 lbs.

"Here the strength to resist crushing follows the ratio of the squares of the depths, as is found to be the case in the transverse fracture of rectangular bodies of constant breadth and span."

I wish here to take exception to the general deductions which Fairbairn has drawn from these experiments. The iron tried, from its thinness, possessed no strength when considered as a beam; it was already strained to an unknown extent, from its contraction in cooling. And, at best, it was only a test of the tensile strength of the material, when ruptured under unfavorable circumstances. Had the distance between the supports been greater, permitting the iron to stretch and bend

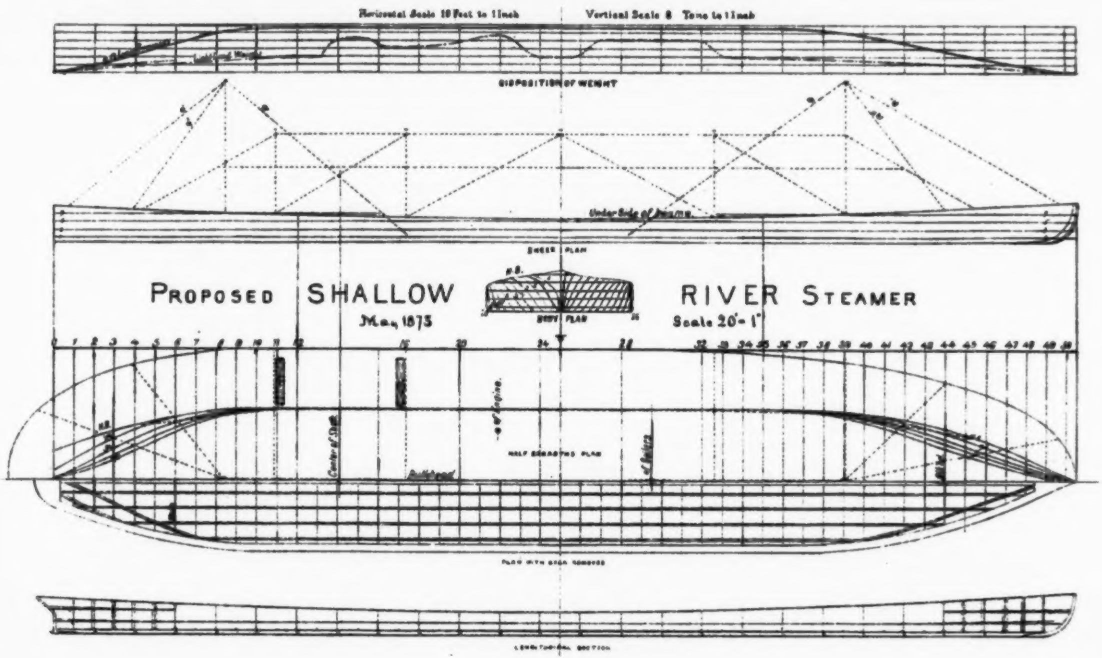
to a greater extent, the results obtained would have been greater. The strain being tensile, with similar quality of material, the bursting would of course vary directly in accordance with the thickness. If, on the other hand, the iron had been sufficiently stiff from increased thickness, or by reason of the supports being closer together, so that it could have rested on the supports unsecured, as was the case with the oak, then I believe the resistance to bursting would have been as the squares of the depths. In experiments on the penetration of shot through armor plates, the resistance has been found to follow this general law, which tends to corroborate the view I have taken.

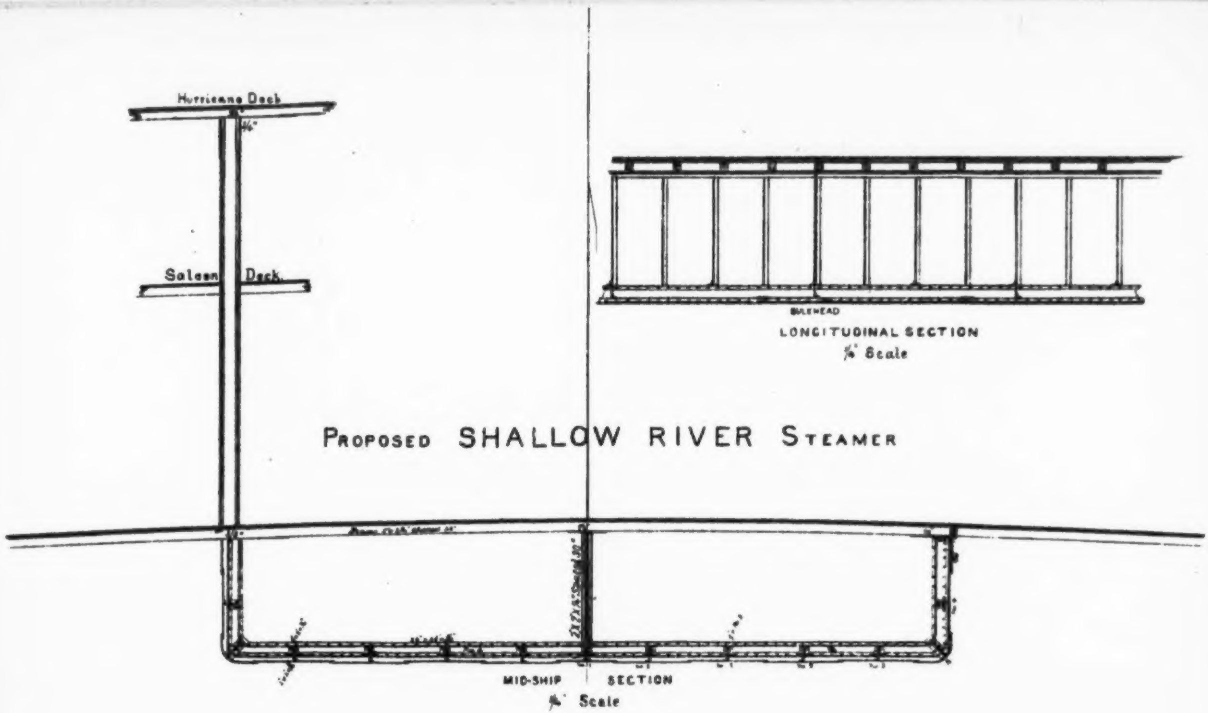
By this experiment it is shown where the distance between the supports is the same, one quarter-inch iron plating is equivalent to 3-inch oak planking. If the frames in the iron hull are farther apart, and the iron not previously strained, the iron will, as I have stated, probably stand a much greater relative strain than the experiments showed.

In the vessel taken for comparison, the planking is of 3½-inch oak, the unsupported distance between the frames being 8 to 9 inches; considering the decay of the oak, its water-soaked condition and the narrow planks of which its sheathing is composed, I have upon the above basis estimated that plate-iron of good quality, ¼-inch thick, will resist as great a punching force as 3½-inch oak planking. In regard to the weight; the oak, when water soaked, as it becomes soon after the vessel is launched, will weigh about 5 lbs. a square foot for each inch in thickness, or 17½ lbs. for the 3½-inch planking; while iron of ¼-inch in thickness weighs but 10 lbs. per square foot, and never increases in weight by use. To obtain the full benefit of the yielding of the bottom plating between the longitudinal frames—hereafter described—when the bottom strikes an obstruction, there must be no point from stem to stern between the longitudinals where the elasticity is destroyed by increase in stiffness, due to the securing of bulkheads or transverse intercostal frames to the skin; *that is, no transverse bulkheads or frames must be permitted to be secured to, or even rest upon the skin of the bottom.* As, however, it is necessary that the longitudinal frames or floors shall be stiffened by transverse bracing to keep them up to their proper position, I introduce, every 10 feet in the length of the hull, a partial bulkhead reaching down to within about 3 inches of the skin. The plate forming this partial bulkhead extends 3 inches above the top of the longitudinal floors, and is secured to their topping angle bars by a bar of angle iron, which by thus riding over the floors may be made continuous in one piece from the central bulkhead to

the beam shelf, effectually tying all the longitudinals together, and preventing their buckling when under compressive strain. To the bottom of the intercostal plate (3 inches above the skin) to stiffen it, is secured a bar of light angle iron, and the ends of the plate are secured to the longitudinals, by vertical corner pieces. Near each end of the boat a transverse water-tight bulkhead is introduced; the continuity of elasticity is preserved by terminating the plates of this bulkhead at 3 inches from the skin, and tightness is obtained by securing to these plates a strong piece of sheet rubber, or other suitable material, made sufficiently long to permit it to be carried for a short distance along the skin of the vessel, to which it is fastened, as well as to the longitudinal floors, by light strips of wood or iron of sufficient strength to resist the pressure of a head of water equal to the depth of hold. Suppose, then, a vessel strikes a snag just sufficiently near the bottom of the bilge strake to permit the vessel to be forced upon it. It meets first the bilge strake, made, as hereafter described, strong enough to withstand the shock; the vessel raises a little and passes on; the light iron of the bottom between the two longitudinal floors most nearly in the line of the obstruction, is sprung upwards; the vessel perhaps raises a little, the snag itself yields a little, in fact everything gives a little, until the vessel has passed over, or the resistance has become too great for her power, and she stops. Every one is aware how difficult it is to punch a hole through metal when it rests upon a yielding substance; put a piece of iron on a soft pine plank, and attempt to punch a hole through it, and you will realize to some extent the conditions I have just described. A man will hammer away a long time before he can drive his sledge through a large smoke-pipe, while one blow would have sufficed to have ruptured similar iron, had it been so secured as to have prevented its yielding under the impact.

In regard to the second consideration: I have assumed the wooden hull to have been so proportioned as to be possessed of sufficient strength to resist all the strains to which a boat is usually subjected. The estimated area of the oak in the bottom, including stringers and keelsons, is 1,650 square inches, allowing the strength of the oak to be as to wrought-iron as 1 to 5, and on account of the butts being unspliced in the oak sheathing, taking $\frac{1}{3}$ of the remaining area as effective for tensile strength (which is more favorable for the wood than experiments have shown—Fairbairn's "Ship Building," page 75), we have then for area of equal strength for the iron $1{,}650 \div (5 \times 3) = 110$ square inches; there must, however, be added to the aggregate area, on account of de-


Horizontal Scale 10 Feet to 1 Inch
Vertical Scale 8 Tons to 1 Inch
DISPOSITION OF WEIGHT
Under Side of Beams
SHEER PLAN
PROPOSED SHALLOW RIVER STEAMER
May, 1873
Scale 20'-1"
BODY PLAN
Center of Shaft
of Engine
HALF BREADTHS PLAN
of Boilers
Bulkhead
PLAN WITH DECK REMOVED
LONGITUDINAL SECTION



PROPOSED SHALLOW RIVER STEAMER

ficiency in strength at the butts; as the butts will be double-riveted, 70 per cent. of the total area may be considered as effective, which makes the total area required for the iron 110÷.70=157 square inches, to give same strength as the wooden boat had when new. I have, however, for additional strength, increased the area about 20 per cent., and in the proposed design have 190 square inches. The decks are alike in both cases, but I have introduced in the iron hull on each side under the beams a shelf or stringer 24 inches wide by ½ inch thick, and a similar one upon the central bulkhead.

For the purpose of obtaining the greatest longitudinal strength, in which river vessels, from their shallowness, are most deficient, the longitudinal system is used, as giving the greatest strength with least weight. In the design under consideration the floors are 9 inches deep, and the angle-bar frame is fitted close to the skin without sliver pieces, the bar being joggled over the butt-straps. These floors run in the center of each strake of bottom plating, the strakes running parallel to the keel plate, and dying out on the bilge strake; the longitudinal floors ending as bottom frames on the bilge keelson. The butts of the plating in bottom and floors should be planed, and should have double riveted straps; the plates and bars should be had in as long lengths as possible, and great care should be taken in disposing the butts of plates and bars. The dimensions and arrangements of the iron are shown in the accompanying "midship section." A central bulkhead of plate-iron has been substituted for the wooden bulkhead; it is made water-tight, and securely riveted to the keel-plate, to the transverse frames or partial bulkheads, and to the stringer plate under beams.

In carrying out the third consideration I have made the bilge strake ½ inch thick, and its curved shape and the deep floor or bilge keelson, formed at right angles to a tangent of 45 degrees, give it great strength to resist any force which may be brought against it. At the ends of the boat running to the bulkhead, additional longitudinal stringers are introduced. The stem and stern pieces will be of forged iron, 2 by 5 inches in section, and should run well back on to the keel-plate, which will be thickened to ⅝ of an inch at the bow, and the garboard and next strake will be thickened up at the bilge strake.

The accompanying plans show the general construction of the hull, the distribution of the weight of the hull and machinery, and the buoyancy line on a draught of 4 feet. It will be noticed on one side of the "midship section" plan, and in dotted lines on the sheer plan, I have

introduced a truss or hog-frame, for the purpose of making the boat more rigid. This truss is formed by erecting upon wooden struts a hollow, bottomless girder at such height that the hurricane deck will rest upon it. The struts are additionally secured and the ties strengthened by placing immediately under the saloon-deck beams two heavy plates of iron. The position of these girders and ties is such that it is believed they will not interfere with the deck-room or state-rooms. As will be noticed on the plan, the beam-shelf is made lighter when the hog-frame is used. The following calculations, based upon Tate's rule, as adopted by Fairbairn (see his "Ship Building,") will show the gain in strength by the use of this truss. Breaking strain of vessel when supported in the middle—S in this case being taken at 20 tons for tensile strength per square inch; without "hog frames":

$$M = \frac{S}{h} I_\circ = \frac{20}{3.6} \times 2{,}710.34 = 1{,}505.7.$$

$$M = \frac{W\,l}{8} = \frac{8\,M}{l} = \frac{1{,}505.7 \times 8}{252} = 478 \text{ tons, or } 1{,}070{,}720 \text{ lbs.}$$

With "hog frames":

$$M = \frac{S}{h} I_\circ = \frac{20}{17.7} \times 42{,}745 = 48{,}299.44.$$

$$M = \frac{W\,l}{8} = \frac{8\,M}{l} = \frac{48{,}299.44 \times 8}{252} = 1{,}533.2 \text{ tons, or } 3{,}436{,}368 \text{ lbs.;}$$

or showing an increase in strength of the trussed hull over the untrussed one of more than 3 to 1.

The vessel chosen for comparison of weight is the side-wheel steamboat "Potomac," built for the Ohio and Memphis trade at Cincinnati, Ohio, in 1870. She was of the following general dimensions:

Length on load water-line....................252 feet 6 inches.
Breadth on load water-line....................36 feet.
Breadth over wheel....................66 feet.
Depth of hold....................6 feet.
Thickness of bottom planking, oak....................$3\frac{1}{2}$ ins.
Thickness of side planking, oak....................$3\frac{1}{2}$ and $2\frac{1}{2}$ ins.
Size of floor timbers forward..3 ins. by $6\frac{1}{2}$ ins,; aft, 3 ins. by 6 ins.
Size of top timbers, oak....................$3\frac{1}{2}$ ins. by 4 ins.
Distance between center of frames....................$10\frac{1}{2}$ ins. to 12 ins.
Has one longitudinal bulkhead on keelson.
Deck beams, oak.....5 ins. by $3\frac{1}{2}$ ins.; spaced 28 ins. bet. centers.
Deck planks, pine....................2 ins. thick.
Draught of water with steam up and no cargo—bow, 2 feet 6 ins.; stern, 2 feet 9 ins.; mean, 2 feet $7\frac{1}{2}$ ins.

Displacement in pounds at 2 feet 7½ ins. draught....1,100,000 lbs.
Distance from bow to center of wheels..................182 feet.
Distance from bow to center of boilers..................105 feet.
Number of boilers.......................................Five.
Size of boilers....................37½ ins. diameter, 28 feet long.
Grate surface..............................37 square feet.
Number of steam cylinders...............................Two.
Diameter of cylinders.......................................24 ins.
Stroke of pistons...7 feet.
Diameter of paddle-wheels.................................32 feet.
Length of paddles..12 feet.
Pressure of steam allowed....................148 lbs. per sq. in.
Average speed against current..........................10 miles.
Cost of "Potomac," new..................................$85,000
Usual life, or limit of service of similar boats.........6 to 8 years.

In the accompanying plans the lines, body plan, sheer plan, etc., apply equally to both the iron and wooden hulls; the same deck, and in fact everything above the beam shelf, or upper edge of hull, will be the same in both cases. The dimensions of the plates and angle bars of which the hull is constructed are given in the midship section; the weight of the wooden hull is from actual data. The dimensions of the vessel remaining the same, the weight of iron in the iron hull will be as follows:

Plate-iron and butt straps	229,000 lbs.
Angle iron	56,200 "
Rivet heads and points	6,672 "
Forgings	1,636 "
Total weight of iron in hull	293,508 lbs.

The weights of the two boats (the machinery and upper works remaining the same) will then be:

	For the iron hull.	For the wooden hull.
Iron hull	147 tons.	270 tons.
Deck	112 "	112 "
Machinery and wheels	93 "	93 "
Water in boilers	20 "	20 "
Joiner work	40 "	40 "
Fuel, fittings, etc	25 "	25 "
	437 tons	560 tons.

Mean draught wooden boat as above...............32 ins.
Mean draught iron boat as above..................26 in.

It must be remembered that the draught of such river vessels should be measured by inches.

Let us assume in round numbers that the iron hull weighs 120 tons less than the wooden hull, and will cost 10 cents a pound, or say $30,000, and we will put the cost of the wooden hull at $15,000 (which is said to be much under its cost). Now, assuming the cost of the wooden vessel complete, at $85,000, the cost of the "Potomac" in 1870, and adding $15,000 for the additional cost of the iron hull, and we have the cost of the iron vessel as $100,000.

The durability of wooden boats on the river, when not destroyed by fire or accident, is variously given as averaging from 6 to 9 years. There is no data upon which to predicate the durability of iron hulls (constantly in fresh water) when well constructed; probably 50 years would not be too great a limit. We will assume, however, that the wooden boat lasts 10 years, and is then sold for $15,000, and that the iron hull lasts 20 years, and is then sold for $5,000 more (on account of the iron in its hull), or $20,000. There must then be allowed an annual depreciation, for the wooden vessel, of ($85,000—$15,000)÷10=$7,000; and for the iron hull, of ($100,000—$20,000)÷20=$4,000.

The cost of repairs will be taken as the same in both vessels, and it will be assumed that the vessels are run on equal draughts, say on a mean load draught of 5 feet. The total displacement, in tons of 2,000 lbs., at that draught, is 1,095 tons.

The amount of freight carried on this draught is:

Iron boat...........................	1,095—437=658 tons.
Wooden boat.......................	1,095—560=535 "
Greater amount of cargo carried by iron boat on 5 feet draught...........................	123 "

or about 23 per cent. more freight on mean load draught than the wooden boat, which it must be remembered costs no additional fuel, machinery or expense to carry. For comparison, we will assume the gross profits, during a single season, are 25 per cent. on the first cost of the wooden boat during the time of its life; and as the iron boat, without additional expense, carries 23 per cent. more, we will add 23 per cent. of the profit of wooden boat to the iron boat's earnings; we have then:

		Wooden boat.			Iron boat.
First cost...........		$85,000	......		$100,000
Gross profit.........	$21,250		......	$26,137	
Annual depreciation..	7,000		......	4,000	
Net profit...........	$14,250			$22,137	

This shows for the iron boat a net profit 50 per cent. greater than the net profit of the wooden boat. It is believed that with the same model and equal draughts, less fuel will be consumed in driving the iron boat, as experiments have shown the friction of the water to be less against iron than against wood.

Some doubt may be expressed as to the durability of the iron hull, and it may be thought I have overrated its period of usefulness. It has, however, been often remarked that where iron structures are in constant use, and subject to continual strain, that no perceptible destruction occurs from oxidation; that machinery, the rails of railways, and bridges where much used, do not rust to any extent, although exposed to the weather; and the same fact has been noticed in iron hulls in fresh water. As mentioned before, iron canal boats have been broken up after 40 years' service in England; the hulls even then having been so sound as to cause comment, (Grantham's "Ship Building"). Iron vessels, in this country, especially on our lakes and rivers, have been too recently introduced to permit us to form more than an estimate of their durability. One or two vessels may be cited as examples. The United States side-wheel steamer "Michigan," on Lake Erie, was launched in 1845, and has been ever since in constant service; much of the service has been extremely hazardous, as she has been employed early in the season to open navigation; and late in the season, when new ice was forming, to assist vessels in reaching a harbor. I was informed, in 1870, by a government officer, that up to that time not a dollar had been expended for repairs to her hull. The propeller "Merchant," built at Buffalo by J. C. & E. T. Evans, was launched in 1862, and was one of the first propellers expressly intended for lake service. Her original cost was $80,000, and she was sold last fall, after 10 years' service, for $82,500; the hull, upon careful examination, was found to be perfectly sound. The purchasers, the past winter, added 30 feet to her length, and she is now in service. Other cases might be mentioned, but it is believed that the time is past for prejudice on this point against iron vessels.

It has, I think, been demonstrated that the iron hull, when properly constructed, as compared with an equally good example of wooden hull, possesses the following advantages: the ability, on account of less weight of hull, to carry 20 per cent. more freight upon the same load draught; a durability of at least twice that of the wooden hull; and an increase of more than 50 per cent. in the commercial profit, owing to the causes stated.

I cannot more fitly close this paper than with an extract from Fairbairn's work on "Canal Steam Navigation," published in London in 1831, or more than 40 years ago. "I shall conclude by observing, that the field for improvement in canal and river steam navigation, appears to me most extensive; and if pursued with proper attention to lightness and strength of iron employed in the construction of boats, and the proper disposition of the material, so as to obtain from a given quantity the greatest possible strength, no one can limit the amount of improvement which may thus be obtained in a few years."

Col. Merrill—I am somewhat familiar with the development of iron ship-building in the West, and I find the principal obstacle to its general introduction is the difficulty of persuading Western steamboat men that an iron boat will last sufficiently long to pay for the additional expense. The case is not parallel to that on the great lakes, because, as you are aware, a lake boat is not expected to touch bottom like a river boat; the latter is often stuck on sand bars, and carries heavy spars to push off with.

To the best of my knowledge, there are but two iron steamboats on Western rivers, and both of these were built in Cincinnati. One, the "Alexander Swift," was built at a reported cost of $65,000, while, if constructed of wood, she would not have cost over $40,000. The firm that built her, intended to use her for towing their barges loaded with iron, ore and coal, and of course they threw their profits into the cost of the boat. The other iron boat, the "John T. Moore," was built by the same firm, and sold to prviate parties, who took her up Red River. I have been informed that during the whole time she has been running, (three or four years) not a cent has been expended for repairs to her hull, while wooden boats in the same trade are constantly on the docks. The Red River is one of the worst for snags in the whole Mississippi Valley, as the existence of the Raft would testify. It will thus be seen that iron boat building in the West is yet in its infancy. Some monitors were built during the war, and a few harbor tugs are now built from time to time, but these are not of the ordinary type of Western steamboats, and are not taken into consideration.

A boat was built at Pittsburgh in 1838, the iron for which, if I am not mistaken, came from England. Probably our manufacturing facilities at that time were not great enough to furnish the plates. When this boat, the "Valley Forge," was launched there was great enthusiasm, and a new era in Western boat building was predicted. But the experi-

ment was not repeated, and the "Valley Forge" went South, and wound up her career in Alabama.

After the war the Government built a number of wooden hull snag boats for cleaning out the snags in Western rivers. These boats may be considered as having two ordinary hulls, connected with each other by a shorter scow placed between them (from midships to the stern), and by a heavy beam at the bows, just above the water surface—the space between the beam and the scow being left open to facilitate the disposal of pieces of snags. The beam is called the butting beam, and its use is to loosen the snag in its bed, the boat being run against it at full speed. After being loosened, the snag is drawn up on the butting beam, and there cut up by steam saws. To give you some idea of the tremendous lifting power required on these boats, I will state that the main hoisting chain is composed of links made out of iron 2 inches in diameter, and yet has often been broken in lifting snags. The chain goes directly from the snag to the hoisting drum. According to Trautwine, its breaking strain should be 84 tons. These boats were originally built with a draught of 4 feet, but they now draw 4½, and on this account do not answer their purpose satisfactorily. They work to greatest advantage in low water, as at that time most of the troublesome snags are visible above water. The boats have also become badly strained by the heavy work performed since they were built.

It has not been stated whether there is any destructive oxidation, but, to the best of my knowledge, there is none in fresh water; the greatest wear would come from grounding.

LXVI.

THE FOUNDATIONS OF THE NEW CAPITOL AT ALBANY, N. Y.

By William J. McAlpine, C. E., Member of the Society,

At the Fifth Annual Convention in Louisville, Ky.,
May 21st and 22d, 1873.

Charles Hermany, C. E., in the Chair.

Owing to the limited time remaining, I will not attempt to read a formal paper on the subject announced, but will, instead, state something of what such a paper would contain.

The excavation for these foundations covered between four and five acres area. The material exposed seemed, at first sight, to be of quite uniform quality; but a critical inspection showed that it was widely different, the weight per cubic foot, in one part of the bed, being 70 pounds and in another 90 pounds; part was quicksand, lacking but one element, that of water, but containing sufficient loam to hold the water in suspension, had it been present.

In laying these foundations, great care was taken to prevent any change in the character of the soil. First, a puddle-wall was carried around the whole to keep the water from being collected and retained. Then coarse gravel was spread 6 inches deep over the whole area, so that, in any case, moisture would be carried off; an endeavor was thus made to secure a uniform degree of saturation forever.

One difficulty in making these foundations arose from the great variation in weight of the structure: thus, one part will weigh 170 tons, and an adjacent one but 31 or 32 tons; it was to go from the greater weight to the lesser one, and impose upon the foundations a uniform amount of pressure per square foot in every part.

Before determining a plan for this work, experiments, under my direction, were made, with considerable care, to establish the supporting power of particular soils; which showed substantially that, under the weight imposed up to 2 tons per square foot, no perceptible displacement resulted; and at about 5 tons the surrounding soil was forced upward. In one place the experiment was made 3 feet below the surface, and in another 6 feet; the trials were to be continued at other depths.

Upon examination I found that from 1 to 5 tons weight per square foot was put upon soils like those here met with. Among the recorded cases is one where Robert Stevenson, in building a bridge, attempted to load the clay (which is particularly described) underneath the piers, with 5 tons per square foot. The piers were 170 to 180 feet high. In the construction, when about 140 feet high, they began to sink, which he prevented by transferring a portion of the load to the adjacent earth. Mr. Stevenson erected, in a very hard, peculiar clay, at Newcastle-on-Tyne, a chimney nearly 300 feet high, which settled, and was partly thrown out of line. A few years after he erected another, with base so that the load was 1½ tons per square foot, and it remained intact. The famous tower of St. Pollax, in Glasgow, the tallest in the world, is, I think, nearly 500 feet high; its weight, on the earth foundation, is about 2 tons per square foot.

LXVII.

NOTE RELATING TO

RUMFORD'S DETERMINATION OF THE MECHANICAL EQUIVALENT OF HEAT.

By Prof. ROBERT H. THURSTON, Member of the Society.

PRESENTED DECEMBER 9TH, 1873.

In his "Sketch of Thermodynamics,"* Prof. Tait gives (p. 44, § 78) a *resume* of the history of the growth of that science, and presents the following as the order of its development:

First.—Newton's grand general statement of the laws of transference of mechanical energy from one body or system to another (1687).

Second.—Davy's proof that heat is a form of energy subject to these laws (1799).

Third.—Rumford's close approximation to a measure of the mechanical equivalent (1798).

Fourth.—Fourier's great work on one form of dissipation of energy (1812).

Fifth.—Carnot's fundamental principles, his cycles of operation, and his tests of a perfect engine (1824).

Sixth.—Thompson's introduction of an absolute thermodynamic scale of thermometry (1848).

Seventh.—Joule's exact determination of the mechanical equivalent of heat, and the general reception of the true theory in consequence of his experiments (1843-9).

Eighth.—The adaptation, by Clausius and Rankine, and subsequently, with greater generality and freedom from hypothesis, by Thompson, of mathematical investigation (partly based on Carnot's methods) to the true theory; the reestablishment of the great second law by Clausius, with Joule's experimental verification of Thompson's general results (1849-51).

Ninth.—Thompson's theory of dissipation (1852).

* Sketch of Thermodynamics, by P. G. Tait, M.A., Edinburgh, 1868.

Here, as elsewhere, the author of the above *resume* states fairly the work done by Rumford, but here, as elsewhere, he places his services second in importance to those of Davy, as well as in their actual influence upon the growth of the science of Thermodynamics, and does not, apparently, consider them comparable to those of Joule.

In an earlier portion of the work (pages 7–9, §§ 13–15), the work of Rumford is correctly described, and a value of the mechanical equivalent is deduced and stated at 940 foot-pounds, the estimate being based on the assumption of 30,000 foot-pounds per minute as the true value of a horse-power. The experiment of Rumford consisted in the measurement of the heat developed by the power employed in boring cannon at the Arsenal, in Munich, and his paper describing the method and giving its results was published in the "Philosophical Transactions of the Royal Society of London" for the year 1798.

After showing that the heat evolved through the agency of friction could not have been derived from any surrounding objects, or by compression of the materials employed or acted upon, he says: "It appears to me to be extremely difficult, if not impossible, to form any distinct idea of anything capable of being excited and communicated in the manner that heat was excited and communicated in these experiments, except it be motion,"* and then goes on to urge a zealous and persistent investigation of the laws governing this motion. Estimating the quantity of heat evolved by a power which, as he states, could easily be exerted by one horse, he makes it equal to the "combustion of 9 wax candles, each ¾ of an inch in diameter."† This heat he also states as equivalent to the elevation of "26.58 pounds of ice-cold water" to the boiling point,‡ or 4784.4 thermal units, and the time occupied "one hundred and fifty minutes."

The "horse-power" used by engineers as a unit of power measurement is 33,000 foot-pounds per minute, but this figure, which was taken by Watt, originally, to represent the "average work of the strongest London draught-horses,"§ is much too high for application in estimation of animal power. It is well known among engineers that two-thirds this figure is a more correct value. Rankine‖ gives, for the average draught-horse 25,920 foot-pounds per minute, or 432 per second, and this value, correct as it probably is for Great Britain, is certainly too high for Bavaria. If the horse-power of Rumford be taken at 25,000 foot-pounds per minute,

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* Abridged Phil. Trans., Vol. XVIII, p. 286. † *Ibid.*, p. 284. ‡ *Ibid.*, p. 283. § Bourne.
‖ Steam Engine and Prime Movers, p. 68.

a value far more likely to be correct than 30,000, as assumed in "Sketches of Thermodynamics," the mechanical equivalent, as deduced from Rumford's experiment, becomes 783.8, differing by only 1.5 per cent. from the value now accepted as determined by Joule a half century later, which is nearer the probably correct value than the result of any other investigation, and is even far more accurate than many results obtained by Joule himself.

Could Rumford have eliminated loss due to evaporation, radiation and conduction, of which loss he was well aware, and to the influence of which he refers, it is very certain that he would have given us a more precise determination of this quantity than even that which is above deduced.

We may then claim for Rumford :

First.—That he was the first to prove the immateriality of heat, and to indicate that it is a form of energy, publishing his conclusions a year before Davy.

Second.—That he first, and nearly a half century before Joule, determined, with almost perfect accuracy, the mechanical equivalent of heat.

Third.—That he is entitled to the sole credit of the experimental discovery of the true nature of heat.

The "second" and "third" of the *resume* quoted should, therefore, be transposed, even if the work of Sir Humphrey Davy should not be deemed simply the supplement of earlier labor and merely corroboratory.

Benjamin Thompson, of Concord, New Hampshire, commonly known as Count Rumford, should be accorded a nobler position and a higher distinction than he has yet been given by writers on Thermodynamics.

LXVIII.

AN ACCOUNT OF THE ERECTION OF A BRIDGE OVER THE DANUBE, NEAR VIENNA.

By W. Howard White, C. E., Member of the Society.

Presented December 27th, 1873.

The bridge over the Danube, at Stadtlaw, near Vienna, on the Austrian State Railway, was erected in 1869–70. It was made up of four spans of lattice truss, each 255 feet long, and was of wrought-iron throughout, as is generally the case, even at the present day, with

German iron bridges. Both upper and lower chords were built up of thicknesses of plate, and both chords were continuous over the piers.

Upon the completion of the abutment upon the Vienna bank and of the two nearer piers, two of the spans, which had been set up complete upon the bank in the prolongation of the bridge line, were pushed out into their position by the power of eight men working gearing of 250 times multiplying power; the spans, weighing 417 tons apiece, or together, 834 tons, were moved at the first operation.

To diminish the weight of the portion which must be carried projecting from the abutment, a beak 98 feet long was attached to the forward end of the advancing structure. This beak was essentially a prolongation of the trusses of right triangular form, the base being vertical and of the same height with the truss, and the long side of the right angle being a prolongation of the lower chord. The sharp projecting angle was rounded off from below in such a way as to facilitate the entrance of the advancing structure upon the rollers placed upon the pier. The end of the beak naturally sagged very considerably below its normal position, just before reaching the pier, and it would seem that it would have been necessary to use power to raise the end to the proper position for entering upon the rollers, or else to have used more than the ordinary impulsive force in shoving the rounded end of the beak upon the same, although nothing of the sort is mentioned. In order to strengthen the projecting portion against the unusual strain it was called upon to bear, temporary uprights were bolted on between the regular ones of the extreme span.

The rolling gear consisted of simple iron rollers, or more properly wheels, three feet in diameter, turning upon journals bearing on iron pillow blocks upon the masonry of the abutments and piers. The rollers were grooved in such a way as to receive the rows of rivets in the lower chords into the grooves, which thus acted as guides to resist lateral motion of the bridge in its outward progress. The lower chord having a different thickness in the middle from that at its ends, and the increase being made by the addition of plates to the under side, it was necessary to maintain the surface flush for resting upon the rollers, by laying plates of different thicknesses at different portions of the chord, between it and the rollers. This plan, however, was found to work so indifferently for steadiness of motion, that it is recommended, in any future use of the method, to make provision for it beforehand, by carrying a portion of the width of the lower chord out for the whole length

of the span of thickness, uniform with the thickest portion of the chord, to serve as a rolling surface ; the increase of strength to the chord being compensated for by diminished width of the same.

On the successful completion of the first operation, the remaining two spans were set up on the shore, directly behind and in connection with the others, and all four spans were pushed out together until the beak projected over the last pier, beyond which were a number of short spans over a high water channel of the river, and the four spans arrived simultaneously in position ; their collected weight, with the beak, being about 1,800 tons. This weight required the use of steam instead of the hand power hitherto used.

The pushing gear was connected with brackets temporarily bolted to the lower chord by means of chains, the brackets being shifted as the motion progressed. In order to bring the bridge to its final position it was necessary to make use of a beak at the rear of the bridge, to which the chains could be attached ; an expense which it would seem might have been avoided by using hydraulic power and shoving action proper, instead of the chain machinery actually employed.

A noticeable feature in the building of this bridge was the fact that no piling whatever was used; the piers being founded hydraulically upon iron caissons, held in position before reaching their bed, by chains from the river bank. A further novel feature in the bridge was the carrying of the rails directly upon longitudinal iron bearers, which was not found to give bad results.

The Creusot (France) Company, who executed the iron-work for this bridge, have turned the lattice form of the truss to advantage in the erection of other bridges. In the celebrated railway bridge at Fribourg, in Switzerland, where two of the piers are 229 feet in height, the iron-work was built out from the shore, or, at all events, from the first pier onwards, by addition to the end which projected freely into the air, and the projecting portion, after each span had reached its complete length, was used as a derrick for the construction of the iron portion of the pier. The lattice truss is very popular throughout Germany and Austria, and many specimens have a very light and elegant appearance.

For comparison of American and German bridges, I can only give the weight of the bridge at Stadtlaw, 3,662 lbs. per foot, and the weight it was tested to carry, 5,375 lbs. per foot. A detailed account of this bridge, with plans, is to be found in Förster's "Allgemeine Bau-Zeitung" for 1870, published in Vienna.

LXIX.

TABLES OF THE STRENGTH OF CAST-IRON COLUMNS.

A Paper by Edwin Thacher, C. E., Member of the Society.

Read at the Fifth Annual Convention, in Louisville, Ky., May 21st and 22d, 1873.

Charles Hermany, C. E., in the Chair.

Hodgkinson's formulæ, as all who have occasion to use them well know, involve laborious calculations, and require much time in their application. No tables giving correct results, and of sufficient range for the purposes of the engineer, so far as the writer knows, have been calculated. The following tables are presented to the Society to enable Mr. Hodgkinson's formulæ to be quickly and accurately applied to any case likely to arise in practice.

Mr. Hodgkinson, in his original paper, published in the "Philosophical Transactions of the Royal Society," 1840, gave the following formulæ, derived from a large number of experiments on "Low Moor cast-iron No. 3":

Solid columns, both ends flat, $W=98922\frac{d^{3.55}}{l^{1.7}}$ (1)

Hollow columns, both ends flat, $W=99318\frac{d^{3.55}-d_1{}^{3.55}}{l^{1.7}}$ (2)

Solid columns, both ends rounded, $W=33379\frac{d^{3.76}}{l^{1.7}}$ (3)

Hollow columns, both ends rounded, $W=29120\frac{d^{3.76}-d_1{}^{3.76}}{l^{1.7}}$ (4)

Formulæ (1) and (2) apply to columns of about 30 diameters and upwards; formulæ (3) and (4) to columns of about 15 diameters and upwards. For solid or hollow cylindrical columns of less than 30 diameters flat at the ends, or less than 15 diameters rounded at the ends, he found the following formulæ to give approximately the correct result. $W=\frac{W\,C}{W+\frac{3}{4}C}$ (5) in which W=breaking weight by preceding formulæ for long columns, and C = crushing strength of the material,

which, for Low Moor cast-iron, No. 3, he found to be 109801 lbs. per square inch.

Some doubt having arisen as to the safety of employing Hodgkinson's formulæ, derived from experiments on Low Moor iron, No. 3, to other varieties of iron, he made a second series of experiments, and in 1857 presented his second paper to the "Royal Society," detailing experiments on 13 kinds of iron from various parts of Great Britain ; 2 of them being mixtures. These induced him to modify his original formulæ to some extent, giving for solid round columns with flat ends $W = m \frac{d^{3.5}}{l^{1.63}}$ (6), in which "m" varies from 75270 lbs. to 111858 lbs., the mean of 13 irons being 95486 lbs. For hollow cylindrical columns of Low Moor iron, No. 2, he gives $W = 94859 \frac{d^{3.5} - d_1^{3.5}}{l^{1.63}}$ (7). Mr. J. B. Francis, in his book on the Strength of Cast-iron Pillars, Tables I, II, and III, has compared Mr. Hodgkinson's experiments of 1856 with the formulæ presented in 1840, with the following results :

1st. "For hollow cylinders of Low Moor iron, No. 2, with flat ends, varying from 2.51 inches to 3.8 inches, external diameter, the breaking weights by experiment were found to vary from 18 per cent. greater to about 1 per cent. less than the computed breaking weights ; the mean being about 8 per cent. greater."

2d. "For solid cylinders with flat ends, cast from 13 kinds of iron in Great Britain, varying from 2.55 inches to 1.54 inches external diameter, the breaking weights by experiment varied from 30 per cent. greater to 15 per cent. less than the computed breaking weights ; the mean being about 9 per cent. greater."

3d. "For solid cylinders with flat ends, from fragments of those several kinds, the breaking weights, by experiment, varied from 33 per cent. greater to 23 per cent. less than the calculated weights ; the mean being 5 per cent. greater."

It is seen from these results that the formulæ first presented, agree quite well with the later experiments. Mr. Francis also compared the breaking weights of hollow cylinders by the experiments of 1856 with each formula, and found that the first formulæ agreed best with the larger columns, and the last formulæ with the smaller columns. As the first formulæ were established from experiments on columns of small diameter, and the later ones after the experiments on those of larger diameter had been made, the results of this comparison are rather

surprising ; yet there seems to be little doubt that for the columns commonly used in practice, the formulæ first given are the safest and best to adopt, and these are the ones from which the following tables are calculated.

According to those formulæ first established by Mr. Hodgkinson, the strength of cylinders with flat ends vary as $\frac{d^{3.55}}{l^{1.7}}$ or $\frac{d^{3.55}-d_1{}^{3.55}}{l^{1.7}}$, and the strength of cylinders with rounded ends vary as $\frac{d^{3.76}}{l^{1.7}}$ or $\frac{d^{3.76}-d_1{}^{3.76}}{l^{1.7}}$. The values of $\frac{l}{d}$ for all similar columns are constant, and these formulæ may be written :

For cylinders with flat ends $\frac{d^{1.85}}{\left(\frac{l}{d}\right)^{1.7}}$ or $\frac{d^{1.85}-d_1{}^{1.85}}{\left(\frac{l}{d}\right)^{1.7}}$.

For cylinders with rounded ends $\frac{d^{2.06}}{\left(\frac{l}{d}\right)^{1.7}}$ or $\frac{d^{2.06}-d_1{}^{2.06}}{\left(\frac{l}{d}\right)^{1.7}}$.

The strength, therefore, of similar cylindrical columns varies as follows :—when the ends are flat, as $d^{1.85}$ or $d^{1.85}-d_1{}^{1.85}$, and when the ends are rounded, as $d^{2.06}$ or $d^{2.06}-d_1{}^{2.06}$. Mr. Hodgkinson found, by comparing such solid cylindrical columns as he was able to collect from his experiments, that the strength of those with flat ends varied as $d^{1.82}$, and those with rounded ends as $d^{1.928}$, these being mean results. The areas of similar columns vary as d^2 or $d^2-d_1{}^2$. It is seen that for similar columns with rounded ends, the formulæ make the strength vary as a power of the diameter a little higher than the square, and the experiments, with a power of the diameter a little lower than the square, whence, for all practical purposes, we may consider the strength to vary as the square of the diameter, or as the area of transverse section. For similar columns with flat ends we found the strength to vary by formulæ as $d^{1.85}$ and $d^{1.85}-d_1{}^{1.85}$, which differs from the square so much that a correction will be applied, which will be explained hereafter.

The following tables have been computed on the principle that the strength of similar columns varies as the areas of their transverse sections, with corrections for flat-ended columns, and those flat at one end and rounded at the other. Hollow columns, in order to be similar, must not only have the ratio of length to diameter constant, but the ratio of diameter to thickness of metal must also be constant. Tables I, II and III give the breaking weights in pounds per square inch of section, of

cylindrical or octagonal columns from 4 to 60 diameters, and for all thicknesses of metal, from solid to one-sixteenth the external diameter, which is a greater range, both as regards length and thickness of metal, than is often used in practice. Table I gives the breaking weights per square inch, of columns flat at the ends. Table II gives the breaking weights per square inch, of columns flat at one end and rounded at the other. Table III gives the breaking weights per square inch, of columns rounded at both ends. Table IV gives the areas of solid or hollow octagonal or cylindrical sections, from 2 inches to 20 inches diameter, and from ¼ inch to 2 inches thickness of metal, which can be used in connection with preceding tables in finding the strength of columns of any required diameter and thickness; the octagonal sections have circular internal diameter.

In Tables I, II, III, the values of "r" given at top of the page are the quotients obtained by dividing outside diameter by twice the thickness of metal:—column 1 gives the ratios of length to diameter from 4 to 60; column 2 gives the breaking weights per square inch, of solid cylinders; columns 3, 4, 5, 6, 7, 8 and 9 give the breaking weights per square inch, of hollow cylinders for different values of "r"; column 10 gives the breaking weights per square inch, of thin cylinders by Gordon's formula, $W = \frac{80000}{1 + \frac{1}{400}\left(\frac{l}{d}\right)^2}$, which admits of direct comparison with those of Hodgkinson; columns 11 and 12 give the corrections to be applied for various diameters when considered necessary.

In calculating these tables, for convenience, a standard column of 12 inches external diameter has been used, and made to assume in succession every condition as regards length and thickness of metal. This gave the advantage of applying the formulæ in the same form in which they are given, "d" being in inches, and "l" in feet, also the additional advantage of using "l" in whole numbers, it being of the same value as $\frac{l}{d}$.

For example, in Table I, the breaking weight per square inch of solid columns $= 98922 \frac{12^{3.55}}{\text{area} \times \left(\frac{l}{d}\right)^{1.7}} = \frac{5928300}{\left(\frac{l}{d}\right)^{1.7}}$, and of hollow columns where r=6, it$=99318 \frac{12^{3.55} - 10^{2.55}}{\text{area} \times \left(\frac{l}{d}\right)^{1.7}} = \frac{9283130}{\left(\frac{l}{d}\right)^{1.7}}$, and in the same way for all others. This paper was intended to

embrace columns of cylindrical and octagonal sections only, these being the most economical and frequent forms used, but, if required, the tables can be used for × and H sections, as follows :—Mr. Hodgkinson found that the ratio of strength which columns of × section bore to hollow cylinders to be as 17578 to 39645, and for H section the ratio was as 29571 to 39645, whence the breaking weights per square inch of columns of × section = tabular strength × 0.45, and the breaking weights per square inch of columns of H section = tabular strength × 0.74.

As the tables give breaking weights, such a factor of safety must be used as in the judgment of the engineer he considers necessary. Comparing the results given by Gordon's formula, in column 10 of the tables, with the strength of hollow cylinders as given by Hodgkinson's formulæ, it will be seen that there is a wide difference between them. In Table I, for columns with flat ends, Gordon's formula gives 75 to 95 per cent. of Hodgkinson's results, and in Table III, for columns with rounded ends, from 46 to 74 per cent. of Hodgkinson's results, depending on length and diameter of column.

Attention is called to the fact, that the ratio of strength of long flat-ended and round ended columns is $\frac{1}{3}$ only for cylinders not exceeding 2 inches in diameter. A long flat-ended column, 12 inches in diameter, is not quite twice as strong as the same column with rounded ends, and the ratio changes with every diameter. The limit between long and short flat-ended columns varies from $\frac{l}{d} = 23$ to $\frac{l}{d} = 31$, and between long and short round-ended columns from $\frac{l}{d} = 16$ to $\frac{l}{d} = 21$; the heavy horizontal lines in the tables indicate this limit.

Corrections.—1. Long columns with flat ends :

1st. When solid.—The ratio of strength which a column of any diameter should bear to one having a diameter of 12 inches is $\left(\frac{d}{12}\right)^{1.85}$; the ratio given by tables is $\left(\frac{d}{12}\right)^{2}$; their quotient multiplied by the tabular number will give the true number, and $\frac{d^{1.85} \times 12^2}{12^{1.85} \times d^2} - 1$ is the percentage of error in the tabular number; which reduced, becomes $\frac{1.452}{d^{0.15}} - 1$.

2d. When hollow.—Let d = external diameter of standard column = 12 inches.

Let d_1 = internal diameter of same.

Let d_2 = external diameter of any other column.

Let d_3 = its internal diameter.

Proceeding in the same manner as above, we have percentage of error $= \frac{(d^{1.85} - d_1^{1.85}) \times (d_2^2 - d_3^2)}{(d_2^{1.85} - d_3^{1.85}) \times (d^2 - d_1^2)} - 1$; but for similar columns $d:d_1::d_2:d_3. \therefore d_3 = \frac{d_1 \, d_2}{d}$, whence substituting, factoring and reducing, this becomes $\frac{1.452}{d^{0.15}} - 1$, the same as before, whence we see that the percentage of error is independent of internal diameter. From this formula the column of corrections in Table I was determined.

2. Long columns, flat at one end and rounded at the other :

The quantities for long columns in Table I are to the corresponding quantities in Table III as $99318 \times 12^{3.55}$ is to $29120 \times 12^{3.76}$, or as 2 to 1 nearly; the correction, therefore, is expressed by $\frac{2}{3}\left[\frac{1.452}{d^{0.15}} - 1\right]$ and is given for columns from 2 to 24 inches diameter in Table II.

3. Short columns with flat ends :

If $\frac{1.452}{d^{0.15}} = A$; correction $= \frac{A\,W\,C}{A\,W + \frac{3}{4}\,C} \times \frac{W + \frac{3}{4}\,C}{W\,C} - 1 = \frac{A\,(W + \frac{3}{4}\,C)}{A\,W + \frac{3}{4}\,C} - 1$, which for a constant diameter is variable ; but I have found this variation to be as follows : when $\frac{l}{d} = 1$, correction $= 0$; when $\frac{l}{d} = 30$, or at the limit between long and short columns wherever it may be, correction $= \frac{1.452}{d^{0.15}}$ as given in the Table, and for any intermediate value of $\frac{l}{d}$, the correction is in direct proportion to that value. For short columns, flat at one end and rounded at the other, between 4 and 30 diameters, the same rule also applies. By applying these corrections the strength, true to the formulas, of any column, whatever be its diameter, length, thickness of metal or condition, can readily be found, but within ordinary limits of diameter the corrections are so slight that they may in most practical cases be neglected.

TABLE I.

BREAKING WEIGHTS PER SQUARE INCH OF CYLINDRICAL OR OCTAGONAL COLUMNS OF CAST-IRON, FLAT AT BOTH ENDS.

L. / D.	Outside diameter divided by twice the thickness of metal = R.								By Gordon's Formula.	Correction.	
	Solid.	2.	3.	4.	5.	6.	7.	8.		Diam.	Per Cent.
4	95910	98216	99403	99993	100340	100566	100729	100839	76923	2	+31.0
5	90559	93586	95173	95966	96434	96739	96958	97107	75294	3	+23.1
6	85095	88773	90729	91715	92298	92679	92954	93141	73394	4	+17.9
7	79691	83923	86207	87365	88053	88505	88831	89054	71269	5	+14.0
8	74458	79138	81700	83009	83790	84304	84675	84929	68966	6	+11.0
9	69484	74506	77294	78728	79588	80154	80565	80846	66528	7	+8.4
10	64801	70070	73033	74569	75494	76104	76547	76850	64000	8	+6.4
11	60436	65868	68959	70574	71548	72193	72663	72985	61420	9	+4.5
12	56384	61906	65086	66757	67769	68441	68930	69266	58823	10	+2.8
13	52644	58197	61430	63138	64178	64868	65372	65719	56239	11	+1.3
14	49200	54737	57993	59723	60779	61483	61996	62349	53691	12	0.0
15	46034	51516	54770	56510	57574	58283	58804	59161	51200	13	—1.2
16	43125	48524	51757	53493	54558	55270	55791	56151	48780	14	—2.3
17	40454	45747	48942	50666	51727	52436	52957	53317	46444	15	—3.3
18	38004	43176	46320	48025	49076	49780	50297	50654	44198	16	—4.2
19	35750	40789	43874	45552	46590	47286	47791	48152	42049	17	—5.1
20	33679	38054	41595	43245	44266	44951	45456	45805	40000	18	—5.9
21	31772	36526	39473	41088	42090	42764	43261	43605	38049	19	—6.6
22	30016	34623	37494	39073	40055	40716	41204	41541	36199	20	—7.3
23	28396	32857	35651	37192	38152	38799	39278	39608	34446	21	—8.0
24	26704	31214	33930	35432	36369	37002	37469	37793	32787	22	—8.7
25	24919	29690	32328	33791	34705	35323	35779	36096	31218	23	—9.2
26	23303	28260	30819	32242	33132	33734	34179	34488	29740	24	—9.9
27	21859	26755	29420	30804	31670	32257	32691	32993	28343	..	....
28	20549	25152	28108	29451	30295	30867	31289	31582	27027	..	....
29	19360	23696	26678	28184	29004	29561	29972	30258	25785	..	....
30	18274	22367	25181	26826	27788	28329	28730	29008	24615	..	....
31	17283	21154	23816	25371	26374	27065	27562	27833	23512	..	....
32	16376	20044	22566	24040	24990	25644	26136	26480	22472	..	....
33	15539	19019	21412	22811	23712	24334	24800	25126	21491	..	....
34	14769	18077	20351	21680	22537	23128	23571	23881	20565	..	....

TABLE I.—(*Continued.*)

BREAKING WEIGHTS PER SQUARE INCH OF CYLINDRICAL OR OCTAGONAL COLUMNS OF CAST-IRON, FLAT AT BOTH ENDS.

L. D.	OUTSIDE DIAMETER DIVIDED BY TWICE THE THICKNESS OF METAL = R.								BY GORDON'S FORMULA.	CORRECTION.	
	SOLID.	2.	3.	4.	5.	6.	7.	8.		DIAM.	PER CENT.
35	14061	17210	19376	20641	21457	22019	22441	22736	19692	..	....
36	13403	16405	18469	19675	20452	20989	21391	21672	18868	..	....
37	12793	15658	17628	18780	19522	20033	20417	20686	18089	..	....
38	12226	14964	16847	17948	18656	19145	19512	19769	17353	..	....
39	11697	14317	16118	17171	17849	18317	18668	18914	16658	..	....
40	11204	13714	15439	16447	17097	17545	17881	18116	16000	..	....
41	10745	13152	14806	15773	16397	16826	17149	17375	15377	..	....
42	10314	12624	14212	15140	15739	16151	16461	16678	14787	..	....
43	9908	12127	13653	14545	15119	15516	15813	16021	14228	..	....
44	9529	11663	13130	13988	14541	14922	15208	15408	13698	..	....
45	9172	11226	12639	13404	13996	14363	14638	14831	13195	..	....
46	8836	10815	12176	12971	13483	13837	14102	14287	12719	..	....
47	8519	10427	11739	12506	13000	13340	13596	13775	12265	..	....
48	8219	10060	11325	12065	12542	12870	13117	13290	11834	..	....
49	7936	9713	10935	11650	12110	12427	12665	12832	11424	..	....
50	7668	9385	10566	11256	11701	12008	12238	12399	11034	..	....
51	7414	9074	10216	10884	11314	11610	11832	11988	10663	..	....
52	7174	8781	9885	10531	10947	11234	11449	11600	10309	..	....
53	6945	8500	9570	10195	10598	10875	11084	11230	9972	..	....
54	6727	8234	9270	9875	10265	10534	10736	10877	9650	..	....
55	6521	7981	8986	9573	9951	10212	10407	10544	9343	..	....
56	6324	7740	8714	9283	9650	9903	10093	10226	9050	..	.
57	6137	7511	8456	9009	9365	9610	9795	9923	8769	..	....
58	5958	7292	8210	8746	9092	9330	9508	9634	8501	..	....
59	5789	7085	7977	8498	8834	9065	9239	9361	8245	..	....
60	5624	6884	7750	8256	8582	8807	8976	9094	8000	..	....

TABLE II.

BREAKING WEIGHTS PER SQUARE INCH, OF CYLINDRICAL OR OCTAGONAL COLUMNS OF CAST-IRON, FLAT AT ONE END AND ROUNDED AT THE OTHER.

L. / D.	Outside diameter divided by twice the thickness of metal = R.								By Gordon's Formula.	Correction.	
	Solid.	2.	3.	4.	5.	6.	7.	8.		Diam.	Per Cent.
4	91652	93461	95138	95998	96472	96795	97034	97191	72944	2	+20.7
5	85194	87510	89650	90756	91370	91790	92101	92305	69647	3	+15.4
6	78844	81593	84116	85434	86170	86674	87048	87295	66108	4	+11.9
7	72787	75882	78704	80192	81029	81605	82032	82315	62480	5	+9.3
8	67114	70468	73513	75131	76047	76679	77148	77459	58873	6	+7.3
9	61881	65419	68613	70325	71300	71973	72474	72807	55363	7	+5.6
10	57086	60739	64028	65800	66817	67519	68043	68392	52000	8	+4.3
11	52723	56438	59767	61577	62620	63341	63877	64240	48809	9	+3.0
12	48759	52489	55825	57648	58704	59435	59982	60348	45805	10	+1.9
13	45169	48880	52191	54011	55070	55804	56354	56722	42989	11	+0.9
14	41919	45585	48848	50651	51705	52436	52984	53352	40359	12	0.0
15	38975	42575	45774	47551	48593	49316	49860	50225	37907	13	—0.8
16	36307	39825	42950	44691	45717	46429	46965	47325	35626	14	—1.5
17	33856	37313	40353	42055	43060	43758	44284	44639	33504	15	—2.2
18	31369	34914	37968	39626	40607	41291	41806	42154	31533	16	—2.8
19	29156	32549	35771	37382	38339	39006	39505	39847	29701	17	—3.4
20	27178	30167	33503	35293	36243	36892	37381	37712	28000	18	—3.9
21	25402	28516	31430	33126	34146	34869	35409	35731	26418	19	—4.4
22	23800	26785	29552	31161	32133	32819	33342	33696	24949	20	—4.9
23	22351	25213	27844	29376	30300	30954	31451	31789	23582	21	—5.3
24	20935	23778	26284	27743	28625	29249	29723	30045	22310	22	—5.8
25	19536	22470	24860	26252	27095	27691	28143	28451	21126	23	—6.1
26	18269	21260	23542	24871	25677	26246	26678	26973	20024	24	—6.6
27	17137	20066	22338	23610	24382	24926	25340	25622	18996	..	....
28	16110	18864	21225	22441	23182	23704	44100	24370	18038	..	....
29	15178	17772	20095	21361	22071	22572	22952	23211	17143	..	....
30	14327	16775	18968	20279	21039	21519	21884	22133	16307	..	....
31	13550	15865	17939	19175	19944	20489	20893	21132	15526	..	....
32	12838	15033	16998	18169	18898	19413	19806	20077	14794	..	...
33	12182	14264	16129	17240	17931	18421	18794	19050	14109	..	....
34	11579	13557	15329	16385	17043	17508	17862	18106	13467	..	...

TABLE II.—(*Continued*).

BREAKING WEIGHTS PER SQUARE INCH, OF CYLINDRICAL OR OCTAGONAL COLUMNS OF CAST-IRON, FLAT AT ONE END AND ROUNDED AT THE OTHER.

L./D.	Outside diameter divided by twice the thickness of metal = R.								By Gordon's Formula.	Correction.	
	Solid.	2.	3.	4.	5.	6.	7.	8.		Diam.	Per Cent.
35	11024	12907	14595	15600	16226	16669	17006	17238	12864	..	....
36	10508	12303	13912	14870	15466	15889	16210	16431	12299	..	....
37	10029	11743	13278	14193	14763	15165	15473	15684	11767	..	....
38	9585	11223	12690	15565	14108	14493	14787	14989	11267	..	....
39	9170	10737	12141	12977	13498	13866	14147	14340	10796	..	....
40	8784	10285	11629	12430	12929	13282	13551	13735	10353		....
41	8424	9864	11153	11921	12399	12738	12996	13173	9934		....
42	8086	9468	10705	11443	11902	12227	12473	12645	9539	..	....
43	7768	9095	10284	10993	11433	11756	11983	12147	9166	..	....
44	7470	8747	9890	10572	10996	11296	11525	11682	8813	..	....
45	7190	8419	9520	10176	10584	10873	11093	11245	8480	..	....
46	6927	8111	9171	9803	10196	10475	10687	10832	8164	..	....
47	6679	7820	8842	9452	9831	10099	10303	10444	7865	..	....
48	6443	7545	8531	9118	9484	9743	9940	10076	7581	..	....
49	6221	7285	8237	8805	9158	9407	9598	9729	7311	..	....
50	6011	7039	7959	8507	8848	9090	9274	9401	7055	..	....
51	5812	6805	7695	8226	8556	8789	8967	9089	6812	.	....
52	5624	6585	7446	7959	8278	8504	8676	8795	6581	..	....
53	5444	6375	7208	7705	8014	8233	8400	8514	6361	..	....
54	5274	6175	6982	7463	7762	7974	8135	8247	6151	..	....
55	5112	5985	6769	7235	7525	7730	7887	7994	5951	..	....
56	4958	5805	6564	7016	7297	7497	7649	7753	5761	..	....
57	4811	5633	6369	6809	7082	7275	7423	7523	5579	..	....
58	4671	5469	6184	6610	6875	7063	7205	7304	5405	..	....
59	4528	5314	6009	6422	6680	6862	7001	7097	5239	..	....
60	4409	5162	5837	6240	6490	6667	6802	6895	5081	..	....

TABLE III.

BREAKING WEIGHTS PER SQUARE INCH, OF CYLINDRICAL OR OCTAGONAL COLUMNS OF CAST-IRON, ROUNDED AT BOTH ENDS.

L. / D.	Outside diameter divided by twice the thickness of metal = R.								By Gordon's Formula.	Correction.	
	Solid.	2.	3.	4.	5.	6.	7.	8.		Diam.	Per Cent.
4	87395	88707	90874	92003	92604	93024	93339	93543	68966	..	....
5	79829	81435	84127	85546	86307	86842	87244	87503	64000	..	....
6	72594	74413	77503	79153	80042	80670	81143	81450	58823	..	....
7	65884	67841	71202	73020	74006	74705	75233	75576	53691	..	....
8	59770	61799	65326	67254	68305	69054	69621	69990	48780	..	....
9	54279	56330	59933	61922	63013	63793	64384	64769	44199	..	....
10	49372	51409	55023	57032	58141	58935	59540	59935	40000	..	....
11	45011	47008	50576	52581	53692	54489	55091	55495	36199	..	....
12	41135	43073	46564	48539	49639	50429	51034	51430	32787	..	....
13	37695	39564	42953	44884	45962	46740	47336	47726	29740	..	....
14	34638	36433	39704	41579	42631	43390	43972	44356	27027	..	....
15	31917	33634	36779	38593	39612	40349	40916	41289	24615	..	...
16	29489	31127	34144	35890	36876	37589	38139	38499	22472	..	....
17	27258	28879	31765	33444	34393	35081	35612	35961	20565	..	....
18	24735	26652	29617	31227	32139	32803	33315	33655	18868	..	....
19	22562	24310	27669	29212	30088	30726	31219	31543	17354	..	....
20	20678	22280	25411	27341	28220	28833	29306	29619	16000	..	....
21	19032	20507	23388	25164	26203	26974	27557	27858	14788	..	....
22	17585	18947	21610	23250	24211	24923	25480	25852	13699	..	....
23	16306	17569	20038	21560	22449	23110	23624	23971	12719	..	...
24	15167	16342	18639	20055	20882	21497	21977	22297	11834	..	....
25	14154	15250	17393	18714	19486	20060	20508	20807	11034	..	....
26	13236	14261	16265	17500	18223	18759	19178	19458	10309	..	....
27	12416	13378	15257	16416	17094	17596	17990	18252	9650	..	....
28	11672	12576	14343	15432	16069	16542	16912	17158	9050	..	....
29	10996	11848	13513	14539	15139	15584	15933	16165	8501	..	....
30	10380	11184	12755	13723	14290	14710	15039	15258	8000	..	....
31	9817	10577	12063	12979	13515	13913	14224	14431	7540	..	....
32	9301	10022	11430	12298	12806	13183	13477	13674	7117	..	....
33	8826	9510	10846	11670	12151	12508	12788	12975	6728	..	....
34	8389	9038	10308	11091	11549	11889	12154	12332	6369	..	....

TABLE III.—(*Continued*).

BREAKING WEIGHTS, PER SQUARE INCH, OF CYLINDRICAL OR OCTAGONAL COLUMNS OF CAST-IRON, ROUNDED AT BOTH ENDS.

$\frac{L.}{D.}$	Outside diameter divided by twice the thickness of metal = R.								By Gordon's Formula.	Correction.	
	Solid.	2.	3.	4.	5.	6.	7.	8.		Diam.	Per Cent.
35	7987	8605	9814	10560	10996	11319	11572	11741	6037	..	....
36	7613	8202	9355	10065	10481	10789	11030	11191	5730	..	....
37	7266	7829	8929	9607	10004	10298	10529	10682	5446	..	....
38	6944	7482	8534	9182	9561	9842	10062	10209	5181	..	....
39	6644	7158	8164	8784	9147	9416	9627	9767	4935	..	....
40	6364	6857	7820	8414	8762	9019	9221	9355	4706	..	....
41	6103	6576	7500	8069	8402	8650	8843	8972	4492	..	....
42	5858	6312	7199	7746	8065	8303	8488	8612	4292	..	....
43	5628	6063	6916	7441	7748	7976	8154	8273	4104	..	....
44	5412	5832	6651	7156	7452	7671	7842	7957	3929	..	....
45	5209	5613	6402	6888	7172	7383	7548	7659	3765	..	....
46	5019	5407	6167	6636	6910	7113	7272	7378	3610	..	....
47	4839	5214	5946	6398	6662	6858	7011	7113	3465	..	....
48	4668	5030	5737	6172	6427	6616	6764	6863	3328	..	....
49	4507	4857	5539	5960	6206	6388	6531	6626	3198	..	....
50	4355	4693	5352	5750	5996	6173	6311	6403	3077	..	....
51	4211	4537	5175	5568	5798	5968	6102	6190	962	..	....
52	4074	4390	5007	5388	5610	5775	5904	5990	2853	..	....
53	3944	4250	4847	5216	5431	5591	5716	5799	2750	..	....
54	3821	4117	4695	5052	5260	5415	5536	5617	2652	..	....
55	3704	3990	4552	4897	5099	5249	5367	5445	2560	..	....
56	3592	3870	4414	4749	4945	5091	5205	5280	2472	..	....
57	3486	3756	4283	4609	4799	4940	5051	5124	2389	..	....
58	3384	3646	4159	4474	4659	4796	4903	4975	2309	..	....
59	3288	3543	4041	4347	4527	4660	4764	4834	2234	..	....
60	3194	3441	3925	4224	4398	4527	4628	4696	2163	..	....

TABLE IV.

AREAS OF CYLINDRICAL AND OCTAGONAL SECTIONS.

Diameter.	Solid.			Hollow—Thickness of Metal.					
				3/8 Inch.			1/2 Inch.		
	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R
2	3.14	3.31	1	1.91	2.08	3	2.36	2.53	2
3	7.07	7.46	1	3.09	3.48	4	3.93	4.32	3
4	12.57	13.26	1	4.27	4.96	5	5.50	6.19	4
5	19.63	20.71	1	5.44	6.52	7	7.06	8.14	5
6	28.27	29.82	1	6.62	8.17	8	8.64	10.19	6
7	38.48	40.59	1	7.80	9.91	9	10.21	12.32	7
8	50.26	53.02	1	8.98	11.74	11	11.78	14.54	8
9	63.62	67.12	1	10 16	13.66	12	13.36	16.86	9
10	78.54	82.86	1	11.34	15.66	13	14.92	19.24	10
11	95.03	100.30	1	12.51	17.74	15	16.49	21.72	11
12	113.10	119.30	1	13.69	19.91	16	18.06	24.28	12
13	132.70	140.00	1	14.87	22.17	17	19.64	26.94	13
14	153.90	162.40	1	16.05	24.52	19	21.20	29.67	14
15	176.70	186.40	1	17.23	26.95	20	22.78	32.50	15
16	201.00	212.10	1	18.41	29.46	21	24.35	35.40	16
17	227.00	239.50	1	19.59	32.07	23	25.92	38.40	17
18	254.50	268.50	1	20.77	34.77	24	27.49	41.49	18
19	283.50	299.10	1	21.94	37.53	25	29.06	44.65	19
20	314.20	331.40	1	23.13	40.41	27	30.63	47.91	20

Diameter.	Hollow—Thickness of Metal.											
	5/8 Inch.			3/4 Inch.			7/8 Inch.			1 Inch.		
	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R
2	......	......	...	......	......	..	......	......	..	......	......	..
3	4.67	5.06	2	5.30	5.69	2	......	......	..	......	......	..
4	6.63	7.32	3	7.66	8.35	3	8.59	9.28	2	9.43	10.12	2
5	8.59	9.67	4	10.01	11.09	3	11.34	12.42	3	12.56	13.64	2
6	10.55	12.10	5	12.37	13.92	4	14.08	15.63	3	15.70	17.25	3
7	12.51	14.62	6	14.72	16.83	5	16.83	18.94	4	18.85	20.96	3
8	14.48	17.24	6	17.08	19.84	5	19.58	22.34	5	21.99	24.75	4
9	16.45	19.95	7	19.44	22.94	6	22.34	25.84	5	25.14	28.64	4
10	18.41	22.73	8	21.80	26.12	7	25.08	29.40	6	28.28	32.60	5
11	20.37	25.60	9	24.16	29.39	7	27.83	33.06	6	31.41	36.64	5
12	22.33	28.55	10	26.50	32.72	8	30.57	36.79	7	34.55	40.77	6
13	24.30	31.60	11	28.86	36.16	9	33.33	40.63	7	37.70	45.00	6
14	26.25	34.72	11	31.21	39.68	9	36.07	44.54	8	40.84	49.31	7
15	28.22	37.94	12	33.57	43.29	10	38.82	48.54	9	43.98	53.70	7
16	30.19	41.24	13	35.93	46.98	11	41.58	52.63	9	47.13	58.18	8
17	32.15	44.63	14	38.29	50.77	11	44.33	56.81	10	50.27	62.75	8
18	34.12	48.12	14	40.65	54.65	12	47.08	61.08	10	53.41	67.41	9
19	36.08	51.67	15	43.00	58.59	13	49.83	65.42	11	56.55	72.14	9
20	38.04	55.32	16	45.36	62.64	13	52.57	69.85	11	59.69	76.97	10

TABLE IV.—(*Continued.*)

AREAS OF CYLINDRICAL AND OCTAGONAL SECTIONS.

Diameter.	Hollow—Thickness of Metal.																				
	1⅛ Inches.			1¼ Inches.			1⅜ Inches.			1½ Inches.			1⅝ Inches.			1¾ Inches.			2 Inches.		
	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R	Cylindrical.	Octagonal.	R
5	13.70	14.78	2	14.72	15.80	2	.....	.....	..	......	.....	..	......	......	..	......	......	..	......	......	
6	17.23	18.78	3	18.65	20.20	2	19.97	21.52	2	21.20	22.75	2	......	......	..	......	......	..	......	......	
7	20.76	22.87	3	22.58	24.69	3	24.29	26.40	3	25.91	28.02	2	27.44	29.55	2	28.86	30.97	2	......	......	
8	24.29	27.05	4	26.51	29.27	3	28.61	31.37	3	30.63	33.39	3	32.54	35.30	2	34.36	37.12	2	37.69	40.45	2
9	27.84	31.34	4	30.44	33.94	4	32.93	36.43	3	35.35	38.85	3	37.65	41.15	3	39.86	43.36	3	43.99	47.49	2
10	31.37	35.69	4	34.37	38.69	4	37.25	41.57	4	40.06	44.38	3	42.76	47.08	3	45.36	49.68	3	50.27	54.59	2
11	34.90	40.13	5	38.28	43.51	4	41.57	46.80	4	44.77	50.00	4	47.86	53.09	3	50.86	56.09	3	56.55	61.78	3
12	38.43	44.65	5	42.21	48.43	5	45.89	52.11	4	49.47	55.69	4	52.96	59.18	4	56.35	62.57	3	62.83	69.05	3
13	41.96	49.26	6	46.14	53.44	5	50.21	57.51	5	54.18	61.48	4	58.06	65.36	4	61.85	69.15	4	69.11	76.41	3
14	45.50	53.97	6	50.06	58.53	6	54.53	63.00	5	58.90	67.37	5	63.17	71.64	4	67.34	75.81	4	75.39	83.86	3
15	49.03	58.75	7	53.99	63.71	6	58.85	68.57	5	63.61	73.33	5	68.28	78.00	5	72.84	82.56	4	81.67	91.39	4
16	52.57	63.62	7	57.92	68.97	6	63.17	74.22	6	68.33	79.38	5	73.38	84.43	5	78.34	89.39	5	87.96	99.10	4
17	56.10	68.58	8	61.85	74.33	7	67.50	79.98	6	73.05	85.53	6	78.49	90.97	5	83.84	96.32	5	94.25	106.70	4
18	59.64	73.64	8	65.78	79.78	7	71.82	85.82	7	77.76	91.76	6	83.59	97.59	6	89.34	103.30	5	100.50	114.50	4
19	63.17	78.76	8	69.71	85.30	8	76.14	91.73	7	82.47	98.06	6	88.70	104.30	6	94.84	110.40	5	106.80	122.40	5
20	66.71	83.99	9	73.63	90.91	8	80.46	97.74	7	87.18	104.50	7	93.81	111.10	6	100.30	117.60	6	113.10	130.40	5

Note.—A comparison of the principles upon which the tables of Mr. Francis and these of Mr. Thacher were computed, gives the following results :

Mr. Francis made all his computations for long columns from the formula of Mr. Hodgkinson for long columns with flat ends :

$$W = 99318 \frac{d^{3.55} - d_1^{3.55}}{l^{1.7}}$$

taking one-third of the result for columns with rounded ends, and one-fifth of this for the safe load (see Mr. Francis' Book, page 20).

For long columns with flat ends, Mr. Thacher used the same formula and factor of safety, but for columns hinged at both ends, he took another formula of Mr. Hodgkinson :

$$W = 29120 \frac{d^{3.76} - d_1^{3.76}}{l^{1.7}}$$

These hypotheses in many instances give very different results, as shown by the following example :

BREAKING WEIGHTS OF CYLINDRICAL COLUMNS, 10 INCHES EXTERNAL DIAMETER (R=5), 1 INCH THICKNESS OF METAL, 28.28 SQUARE INCHES AREA OF SECTION.

Length in Feet.	$\frac{l}{d}$	Both ends Hinged.				Both ends Flat.			
		Thacher.	Francis.	Ratio.		Thacher.	Francis.	Ratio.	
8	9.6	1699345	731800	2.32	Mean 1.85	2181264	2195400	0.993	Mean 1.00
10	12.0	1403790	644700	2.18		1948887	1934100	1.007	
15	18.0	908890	469100	1.94		1411455	1407300	1.003	
20	24.0	590543	348800	1.69		1051139	1046400	1.002	
25	30.0	404121	270100	1.49		807846	810300	0.997	
30	36.0	296402	198100	1.49		594558	594300	1.000	

It may be noticed that Mr. Thacher's tables give result for a greater range of dimensions than do Mr. Francis' tables, and in pounds per square inch of section.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INCORPORATED 1852.

TRANSACTIONS.

LXX.

FOUNDATIONS UNDER WATER.

A Paper by GABRIEL JORDAN, C. E., Member of the Society.

READ AT THE FIFTH ANNUAL CONVENTION, IN LOUISVILLE, KY., MAY 21ST AND 22D, 1873.

CHARLES HERMANY, C. E., IN THE CHAIR.

Within the last three years, the writer has had occasion to construct a bridge for the Mobile and Montgomery Railroad Company, across Tensas river, in the State of Alabama. The substructure consisted of 12 piers of 2 cylinders each, and a draw-pier of 8 cylinders; the superstructure was composed of 12 spans of 152 feet each, and a draw-span of 260 feet, on the plan of a triangular truss (Fink's improvement).

Tensas river is the largest of the many outlets or bayous of the Mobile river; it is 2,100 feet wide, and from 16 to 35 feet deep, with a daily tidal action of only about 16 inches. The bottom of the river to a very great depth is formed of a light shifting sand, subject to deep and troublesome scouring with the slightest contraction or disturbance of the water way.

The means of the railroad company were too limited to admit of the pneumatic process, and other plans practiced in this country presented the double objection of expense and tendency to produce serious scour. After carefully investigating all the conditions incident to the work, it was determined to use wooden pile piers, incased in cast-iron cylinders.

The piles were driven in two clusters of 12 piles each, the cylinders were then lowered over them and filled with concrete—the whole work presenting very much the appearance of the ordinary pneumatic piers built in many parts of our country. The piles were driven to a depth of 30 feet into the sand, with an ordinary floating steam machine. The first pile in each cluster always drove with ease, and would yield from 2 to 3 inches under the last blow of the hammer, but as the number of piles increased, the resistance increased in a rapid ratio, until no appreciable

effect could be produced on the last pile of each cluster, after reaching the required depth. After the driving of each cluster was complete, the piles were thoroughly bolted together with 1¼-inch bolts, and all sawed off near the surface of low-water.

The cylinders (6 feet in diameter, 1¼ inches thick, in sections of 10 feet, connected by inside flanges and bolts) were lowered into position over the piles to the sand bottom. The work of handling the cylinders was all done from a floating derrick, with heavy blocks and lines and the portable engine used in driving the piles.

Attached to the derrick boat was a small rotary pump, taking its water through a 4-inch suction pipe immediately from the river, and run at a speed of from 200 to 300 revolutions by the same engine used for the other work. The discharge pipe was made of very heavy canvas hose, 3 inches in diameter, about 50 feet long; it led to a cast-iron cone about 10 inches in diameter, from which radiated 12 gas-pipes, 1 inch in diameter and about 2½ feet long. At the end of each of the 12 short pipes was attached a right-angled elbow, and to each was connected a pipe leading down into the cylinder to be sunk. These pipes were made in sections about 10 feet long, so as to lengthen them to any extent, according to the depth of work required. This little apparatus was lowered and raised at will, with a light block and line. As soon as the pipes were lowered into position on the sand, and the pump put in motion, they would sink with great rapidity, very often falling as much as 10 and 15 feet at a single impulse. The sand would at once be put in an active state of ebullition, thus destroying the friction, and the great cylinders would quietly and sometimes quickly sink to the required depth of 15 feet below the bed of the river. The movement of the cylinders was not uniform, but varied with the nature and density of the material passed through; it often required several hours to sink one a single foot. In sinking a cylinder of 4 feet diameter, in the draw-pier, under the immediate supervision of the writer, it was carried to a depth of 14 feet in about two hours; the remaining foot was overcome only after five or six hours' hard pumping and labor. The resistance came from a small deposit of clay, which was finally removed or scoured away by using a single pipe and jet of water on the outside of the cylinder.

While sinking one of the large cylinders, the engineer in charge telegraphed to the general office, that he had encountered a car-wheel about 2 feet under the sand, which had been used as an anchor for a buoy, and that he had no appliances for removing it. While a plan for relief was being considered, and within less than one hour's time, a second

dispatch was received announcing the fact that a single pipe and jet had been passed through the hub of the wheel, which thus had been pumped 16 feet below the surface of the sand, entirely out of the way.

All the fixtures for doing this heavy work were of the rudest character, and very inexpensive—everything was done from the decks of two ordinary flat boats, without staging of any kind. The derricks, pumps and other appliances were of the simplest construction, because the contractor had serious doubts as to the success of the plan, and was unwilling to incur the expense of efficient and well-designed machinery upon an uncertainty. He commenced and carried the work to a successful completion without addition to his immature contrivances.

After the cylinders were sunk to grade they were filled with good shell concrete, deposited through a tube, used somewhat after the manner frequently described in connection with other works in this country; after shrinkage and refilling, a heavy cast-iron cap was bolted over the top of each cylinder, and the piers were complete for the superstructure.

These piers have now been in use about eighteen months, and so far have given entire satisfaction. A uniform scour of about 6 feet occurred at each pier, in curve-shape, extending some 20 feet from the pier; beyond this distance there was no appreciable disturbance. The bottom around the piers was restored to its normal condition by filling the curve with broken stone.

It is not claimed that this process for sinking cylinders or piles can be advantageously applied under conditions differing very materially from those existing at Tensas, as the resistance was found quite difficult to overcome, when small deposits of clay were encountered; and in the event of striking logs or other hard substances, not discovered before locating the piers, the process offers no reliable plan for removing them. Neither is it claimed that the process is new, for it is well known to the profession that many years ago an English engineer resorted to this method of sinking cast-iron piles in the piers of a bridge built over the river Leven for the Ulverstone and Lancaster Railroad Company. During the late war between the States, the Confederate Engineers successfully used the process in sinking heavy wooden piles in the Bay of Mobile. In many instances these piles were driven 10 and 15 feet in the short space of one minute, through a material that could not be penetrated by piles driven in the usual way.

It is believed this method of sinking *large cylinders* has never before been employed or considered to any extent, and it is now brought before the Society, in the belief that it is susceptible of great improvement and

enlarged application, and that it is peculiarly valuable because of the rapidity with which piers may be sunk with it.

Mr. Macdonald—The piers described in this paper are identical in all essential respects with those invented by the late Samuel B. Cushing, member of this Society, and first introduced to practical notice in the bridge at India Point, Providence, R. I., in 1868.* A full description of the method of construction is contained in a pamphlet issued by Mr. Cushing in 1870.

It will be observed that those piers consist of a cluster of timber piles, driven as closely together as possible, in the form of a circle, and enclosed by a cast-iron cylinder, into which is poured a sufficient quantity of concrete to fill all intervening spaces, and completely cover the tops of the piles, the better to protect them from decay above water, and the action of worms below.

We are thus particular in describing the essential principle involved in the use of these piers, as it would appear from the care taken to overcome the difficulties in sinking the encasing cylinders that they and not the enclosed timber piles, contributed the controlling feature of interest to the undertaking. For all purposes of immediate stability, these cylinders might have been omitted. Their sole object, as claimed by the inventor, is by means of the concrete filling to protect the timber piles from decay or deterioration, as must be evident from the present example, when they were driven but 15 feet into a shifting sand, and rested upon the same material.

We do not agree with the writer of this paper that this system cannot be advantageously used under conditions differing materially from those existing at Tensas—the reason he assigns being that it is difficult to sink the cylinders when deposits of clay are encountered. From what has been said of the office performed by the timber piles, it will at once appear that a stiff clay bottom is peculiarly adapted to the placing of Cushing's pile piers. Into such material, clusters of piles may be firmly driven precisely as he has described at Tensas, and the encasing cylinders may then rest almost upon the surface of the bottom, or at most be settled into it a foot or two by their own weight; an additional security is obtained by placing a few feet of rip-rap around the base of the pier. In the case referred to, at India Point, the cylinders were worked into the mud bottom only about 7 feet, and at the Connecticut river bridge a still smaller distance.

* As mentioned in the Memoir of that eminent engineer, in December number of Transactions.

LXXI.

THE CAUSES OF THE FORMATION OF BARS AT THE MOUTHS OF RIVERS, AS SHOWN IN AN EXAMINATION OF THE CONNECTICUT RIVER.

A paper by Gen. THEODORE G. ELLIS, C. E., Member of the Society.

READ AT THE FIFTH ANNUAL CONVENTION HELD IN LOUISVILLE, KY., MAY 21ST AND 22D, 1873.

CHARLES HERMANY, C. E., IN THE CHAIR.

The Connecticut river rises in the extreme northern part of New Hampshire, almost upon the border of Canada, and flows southward between the states of New Hampshire and Vermont, crossing the states of Massachusetts and Connecticut, and enters Long Island Sound.

In the states of New Hampshire and Vermont, its watershed is narrow and precipitous, and its tributaries are mostly small, partaking of the character of mountain torrents. In Massachusetts the drainage area is more extended, and streams of some size enter from the east and west. In Connecticut but one stream of any size enters the river. Below Hartford the tributaries are small, and, except in heavy rains, furnish but little water. All of the streams flowing into the Connecticut bring down more or less detritus in freshets.

From its source to the Massachusetts State line, the banks of the Connecticut are generally of a permanent character. Through the state of Massachusetts the river passes mostly through an alluvial formation, though in some places the bed is hard and permanent. From the town of Northampton to Hartford, and southward as far as Rocky Hill, the river flows through alluvial meadows overflowed in high freshets. In many places through this region the banks are washed away by the river. From about two miles above Hartford to nine miles below, the banks are generally a clay-loam upon the outside of the curves or bends, and upon the opposite side are low sand beaches, deposited by the river in freshets. The banks upon the outside of the bends are wearing away rapidly, generally caving off by being undermined, but in same cases the clay slides out from the bottom and rises up in the river, the top sinking down for

20 or 30 feet back to a level of 8 or 10 feet lower than before. The part that slides out soon washes away, and the operation is repeated. This part of the river furnishes a great amount of silt, and is continually changing its bed.

From Rocky Hill to the Narrows, below Middletown, the banks are more permanent and less subject to wash. At two or three places, however, they are wearing away by the action of the water. Through the Narrows, for about a mile, the banks are high and rocky, and the channel deep and narrow. From this point to the mouth of the river the banks suffer but litte abrasion from the action of the water, and are generally hilly, sloping downward to the water, and in many places rocky. The bed of the river, in the upper part, where it does not reach the rock, is coarse gravel; through the alluvial region it is generally a hard, fine sand, which becomes mixed with more or less mud in the lower part of the river near its mouth.

The Connecticut river is subject to freshets of considerable height, which mostly occur in the spring, when the river is swollen by the melting snow, although freshets have occurred in every month of the year, except June, July and September. The highest known freshet was in May, 1854, when the water rose to a height of 29 feet and 10 inches above low-water at Hartford.

The great height of the freshets at Hartford is due to the contraction of the water-way at the Narrows, just below Middletown. The river at this place is only 650 feet wide at the ordinary water line, with steep and rocky banks rising to a great height on either side, so that the width probably does not exceed 800 feet at the highest water level. Through this gorge all the water must pass which in freshets overflows the banks above and spreads out in some places nearly two miles in width over the meadows through which the river runs. The effect of this contraction is seen in the small fall in the surface of the water in floods between Hartford and Middletown. Middletown is about one-third of the distance from Hartford to the mouth of the river, and the average fall of water between these points in high freshets is only about one-sixth of the total fall. The distance from Hartford to Saybrook light is 49 miles.

The volume of water discharged by the Connecticut river is about 7,500 cubic feet per second at low stages of the water, and rises as high as 160,000 cubic feet per second in high freshets. The average discharge during the year 1871 was 19,388 cubic feet per second. The average velocity of the water varies from one to four miles per hour, according to the stage of the river.

During the spring freshets, the upper waters and tributaries of the Connecticut bring down large quantities of silt, which is deposited along its course, and upon Saybrook bar at its mouth. At high stages a large amount is deposited upon the meadows above Middletown, where the river spreads out to a great width, and the current is checked. Bars of sand are formed in the eddies, which wash away as the river falls. The finer particles held in suspension are carried forward by the current and deposited lower down or carried out over the bar at the mouth of the river. After the water has fallen, so as to be confined within its banks, through the alluvial region between Northampton and Rocky Hill, these banks are washed and carried off; furnishing a great amount of material to be deposited below.

At ordinary summer level, the tides affect the river to a point about seven miles above Hartford, the rise and fall at Hartford being about 10 inches; at Saybrook the mean rise and fall of the tides is 3½ feet. As the water rises the tides disappear. At 5 feet there is no tide at Hartford. In extremely low tides and a south wind, there is sometimes a reverse current as high up as Middletown. The slope of the river from Hartford to the sound, at ordinary summer level and mean tides, is about 2 feet. Above Hartford the slope becomes steeper and more irregular.

The foregoing details of the character of the river, although briefly stated, will give a sufficient idea of the nature and amount of the detritus that may be expected to flow out at the mouth and be deposited in the sound.

In order to know what becomes of the material brought down by the river, it becomes necessary to investigate the currents and other natural causes affecting it after it leaves the river and enters the sound. The nature of the tides and tidal currents in Long Island Sound may be stated briefly as follows: The tidal wave from the ocean approaches the entrance to the sound from the southeast and flows into it, past Fisher's and Plum Islands, mostly through the middle part, called the Race. Another tidal wave enters the west end of the sound through New York Bay. These tides meet at some place between Sand's Point and Throg's Neck, according to the state of the tides. The wave passing up the sound from the Race is partly a wave of transmission, with a progressive velocity depending upon the mean depth of the channel, and partly an actual flowing in and out of water through the Race. The crest of the wave of transmission moves from the Race to Sand's Point, a distance of 95 miles, in two hours. As the tide rises, the water pours through the Race at a maximum rate of

about 5 miles an hour. This current opposite the mouth of the Connecticut river is about 2½ miles an hour. As the crest of the wave passes, the currents set back again in the opposite direction with about the same velocities.

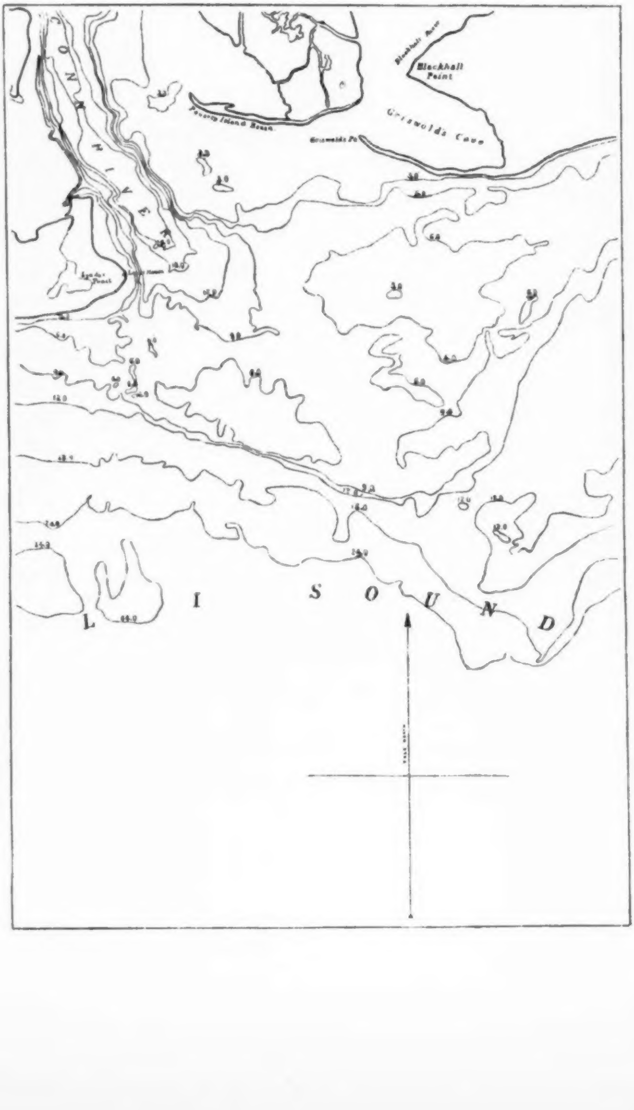
As the tide rises in the sound, the currents set in towards the shore; and as it lowers they set outward from the shore, taking a direction slightly diagonal to the general direction of the central current.

At dead low-water the current from the river spreads out over the whole bar. The greater volume, however, follows the central or southeast channel. There are two other channels over the bar, one to the eastward, near the shore, and another to the southwest, with nearly as deep water as the central channel. When the rising tide commences to flow in from the southeast, it first dams back the river water and forms a distinct line of ripples along the outer edge of the bar. As the tide continues to rise, the stronger flow sets this ripple farther and farther on to the bar; turning back the current from the river, and throwing it farther to the westward, until what still flows out passes through the southwest channel.

From half-tide upward, a strong current sets in along the shore towards Griswold's Point, and flows across the mouth of the river, and partly into it, setting the fresh water back and raising the height of the water in the river. Just before the highest water, the current turns and sets strongly to the eastward along the shore, west from the light-house, impinging strongly upon the current flowing out from the river, and forming a well-defined ripple, extending southwest from the light. The great volume of water coming from the river, that has flowed into it on the rising tide, maintains its current southward until the tide has somewhat fallen, when the great volume of water sweeping down the sound overcomes the obstacle and pushes the river current over to the eastward, into and beyond the main central channel. This action of the littoral currents is evidently what forms the two side channels to the east and southwest.

Along the southwest edge of the bar, the outer bank is for a long distance very steep. This is probably caused by the strong wash of the ebb tide flowing along the shore from the westward, and striking upon the outer edge of the shoal.

A very complete survey of the mouth of the Connecticut river has been recently made by the writer, with a view to permanently improving the channel, which shows with great exactness the present form and state of the bar. By comparison of this with the maps of earlier surveys, we are enabled to determine the changes that have been going on, and study



Blackhall Point
Griswold's Cove
L I S O U N D

the causes which produce them. It will be unnecessary to go into all the details of changes which have occurred from time to time, as it would consume too much space, and prove uninteresting without having copies of the several maps to refer to. The principal changes only, and those having reference to the formation of the bar, will be noticed. A sketch of the mouth of the river in outline is given to make the references more plain to those unacquainted with the localities.

A comparison of the present survey with the United States Coast Survey Chart of 1853, and with a survey made in 1836, by Col. Julius W. Adams, will show the principal changes that have occurred during the past thirty-six years at and near the mouth of the river. Prior to September, 1815, as shown on Col. Adams's map, Griswold's Point extended much farther to the west than at present; reaching some 400 feet beyond what is now known as Poverty Island. To the north, between Griswold's Point and the foot of the island, was the channel of Blackhall River. In continuation of Griswold's Point, with a channel of about 200 feet between, was an island of above 2,300 feet long and 700 feet broad, of an oval form, extending out nearly to Griswold's north pier. The Blackhall river had two outlets, one on each side of the easterly end of the island.

In the gale which occurred in September, 1815, the whole of this island and the western portion of Griswold's Point, back to a long distance beyond its present limits, were washed away. The ordinary action of the waves and currents again commenced to reform the parts carried off, and, at the time of Col. Adams's survey, a point called Poverty Island beach had run out about 1,500 feet from the southwest corner of the large island now called Poverty Island, upon the ground formerly occupied by the island washed away; and Griswold's Point had extended out to within about 1,000 feet of its present limits. The mouth of Blackhall river was then just west of this point.

At the time of the coast survey, in 1850, Poverty Island beach had extended out still farther, and more to the north, to a length of 2,200 feet, and deeper water existed around it than before, Griswold's Point had also lengthened out about 600 feet. Poverty Island beach extended out some 200 feet farther, and was afterwards gradually washed away; being worn down by the action of the waves or river floods, and perhaps both; leaving at the present time but a small island of about 600 feet long, now known as Poverty Island Point. Griswold's Point has to the present time farther lengthened out about 500 feet.

The general outlines of the bar, as shown by the three surveys, are nearly the same. Although the outlines as shown by Col. Adams are not

so extensive as those of the Coast Survey chart and the present survey, yet the two latter do not essentially differ, as to the extent of the bar, at the 18 feet contour line. There have been other minor changes, but none that affect the general character or the causes of formation of the bar.

It will be seen from the foregoing account of the principal changes at the mouth of the river, that great changes are constantly going on upon the eastern side, in the continual formation and occasional washing away of extensive spits and islands, while the western side remains always the same. The cause of this is to be found in the action of the waves during the continuance of the prevailing winds and storms. These are from the southeast, and the waves, acting diagonally along the shore, drive the material of which the beach is formed to the westward, until it meets the descending current of the river, which limits its farther motion in that direction. This wave action is constantly going on, and all the material carried forward eventually finds its way out upon the bar, either by being gradually transferred by the river current or by being washed out at once in a large quantity, as was the case when the island and point were carried off by the gale of September, 1815.

There is another important action of the waves besides that which transmits material along the shore; that is its action upon the outer edge of the bar. It is well known to all who have had occasion to examine the action of the sea upon an inclined shore, that it never carries off or swallows up anything. Everything is cast ashore or pushed out upon the beach, and the largest and heaviest things are thrown the farthest out and highest up upon the beach. This is caused by the progressive movement of the waves as they roll in upon the shore, and the same action occurs upon the outer edge of a bar to a depth equal to that of the same wave action upon the shore. The material of the bar is being constantly forced backward, up to a point where the river current is sufficiently strong to prevent its farther motion, and neutralize the action of the waves.

This wave action upon the bottom has been advanced as the sole cause of bars; the only conditions necessary being :

1st. The presence of sand or shingle, or other easily moved material.

2d. Water of a depth so limited that the waves, during storms, may act on the bottom.

3d. Such an exposure as shall allow of waves being generated of sufficient size to operate on the submerged material.

Where these conditions exist at the mouth of a river running into

the sea or large lake of fresh water, a bar would undoubtedly exist, even though no sediment was discharged by the stream; but there are other causes which form bars, where one of the conditions above might not exist, viz., the limited depth of water before the commencement of the formation of the bar. In this case the action of the waves along the shore carrying sand or shingle into the mouth of the river, as it does in the Connecticut, would soon raise the height of the bottom so that it could be operated upon by the waves and raised into a bar, or the material might be supplied by suspended silt, or sand carried by the current along the bottom, which would be deposited in the dead water at the mouth of the river, and be formed into a bar in the same manner.

When a river brings down silt suspended in the water, a favorite theory has been to suppose that the bar is formed by the deposit of this silt when the fresh water meets the salt. The silt is supposed to remain suspended until the current is checked by the fresh water expanding out over the surface of the underlying salt water, when it is deposited and is carried back by the under current of salt water as the tide enters the mouth of the river, thus forming the bar. The principal objection to this theory is, that bars are formed where streams enter fresh water lakes, and the supposed conditions do not exist.

Another theory of the formation of bars at the mouths of rivers, which has been advanced by the highest authority upon river hydraulics, is, that the sand and mud which is drifted along the bottom by the river current, passes into the sea until it meets the dead angle formed by the rising of the fresh river water over the salt water, when it is deposited and forms the bar. It is not explained what would be the effect of a river flowing into a fresh water lake, where all the water was of the same density; but for a stream flowing into salt water, this theory is amply sufficient to account for the formation of a bar. In the bar upon which observations were made to determine the cause of its formation by experiment, this was found to be the case.

I am led, however, to believe, from an examination of the Connecticut, and comparison with published records of other rivers and estuaries, that no single one of the theories named will account for the formation of all bars. In many cases a variety of causes are in operation to produce a bar and modify its form, either one of which might have produced it acting alone, but which all conduce to give it its existing form. In other cases fewer causes operate to create the bar, and may be different in different bars. In the Connecticut we have many forces in operation to give the bar its present form.

Its material appears to be largely composed of detritus brought down by the river in times of freshets, although some portion of it is washed from along the shore to the eastward by the action of the waves, and some may be swept across the mouth of the river by the littoral tidal currents of the sound. The silt brought down by the river during freshets, and the coarser material that is swept along the bottom are carried out and deposited upon the bar, where their character is modified by the action of the waves and by the tidal currents flowing in and out over the bar when the water in the river falls to its usual level, so that only the coarser part remains, the finer particles of slit being swept away. That part of the bar which remains nearly permanent, or subject to gradual and progressive changes, is mainly composed of coarse sand, much more difficult to move than the ordinary deposits which are formed in the lower portion of the river bed. The form of the outer edge of the bar is probably modified by the strong littoral tidal currents which flow back and forth in this part of the sound. The effect of the current from the westward, however, is more marked than that from the eastward, as it seems to limit the extension of the bar in a southwesterly direction by eroding its outer edge.

From the foregoing, and what has been previously said regarding the wave action along the beach to the eastward, and the action of the tidal currents in forming the side channels over the crest of the bar, it will be seen that we have the following general causes operating to create the bar at Saybrook and modify its form:

1. The suspended silt brought down by the river in freshets, which deposits upon the bar by the decrease of velocity in the stream.

2. The sand and coarse material which is swept along the bottom, and rolls out upon the bar until it meets the water of the sound.

3. The wave action which carries material along the beach towards Griswold's Point, to be in time carried out upon the bar by the river current.

4. The wave action affecting the outer slope of the bar to modify its form and drive back the material.

5. The littoral tidal currents flowing up and down the sound, and the current flowing in and out of the mouth of the river, which erode certain parts of the bar, and probably deposit material upon other parts, and otherwise shift and assort the material brought down by the river, washing away the lighter particles, and thereby modify the general form and character of the bar.

The decrease of velocity in the river water named above may be either

on account of the rising and spreading of the fresh water over the substratum of salt water, or over the crest of the bar itself; although the latter may be considered as a consequence of the bar rather than as an original cause of its formation.

At Saybrook there seems to be two classes of causes operating conjointly to maintain the bar. One class furnishes the material, viz., the suspended silt, the sand washed along the bottom, and the material carried along the beach to the mouth of the river by the wave action. The other class retains the materials upon the bar, viz., the rising of the fresh water over the salt, and forming a dead angle, the spreading out of the water over the bar already formed, and consequent checking of the velocity, and the wave action upon the outer slope of the bar.

It is not presumed that all of the above causes operate upon every bar; one of each class, acting together, would undoubtedly create and maintain a bar. They are named to show that all bars may not be produced by the same causes, nor be accounted for by the same general simple theory. Examples of the formation of bars from different causes may be seen in the deposits at the mouths of the river Wear, at Sunderland, England, and of the Dornoch Firth, in the north of Scotland; which are almost entirely due to the wave action of the sea; and in the vast accumulations at the mouths of the Mississippi and Danube, which are wholly formed of detritus brought down by those rivers. The bar at the mouth of the Wear is shown by dredgings to be formed of the same material as the adjacent beaches, viz., sharp, gritty sand, brickbats, chalk, flint, pebbles, and marly rock; materials which could not by any possibility have come from the river. The bar at the mouth of the Dornoch Firth is fourteen miles to seaward of where the fresh water joins the salt, and is composed of a sharp sand, while the small rivers which discharge into the firth bring down very little silt, and that is of an alluvial character. The material of which the bar is formed is evidently washed from along the shore by the progressive action of the waves from the north sea, with its inner slope modified by the currents from the firth, and its outer slope by the wave action throwing it back. The bar at the mouth of the Mississippi is, without doubt, wholly formed of the sand and mud carried by the current along the bottom, and deposited in the dead angle between the fresh and salt water, as has been described; and the same may be said of the Danube, except that the immense amount of suspended silt brought down by that river into the tideless Black Sea has, perhaps, a more important part in the formation of the bar than the suspended material of the Mississippi, which is carried farther out to sea.

I think the wave action upon the outer slope of bars at the outlets of rivers has a greater effect in modifying their form than is generally supposed. An examination of the outer slope of such bars shows an inclination about that assumed by a beach of the same materials to a certain depth, which is generally that to which the water is affected by waves. The slope then makes an angle, and becomes much flatter. This flatter portion is probably below the influence of the waves, and the sediment lies where it is deposited. This is seen in the form of the Saybrook bar, where there is a flat portion at about 12 feet in depth; in the Mississippi bar, where it is about 100 feet in depth, and also in the sections of other bars of which surveys have been made. The steeper part of the slope just over the crest is evidently not formed in the same manner as the steep lower side of river bars, lying above where they are affected by the tide, in which the slope is formed by the particles of sand which have been rolled to the crest by the current, and drop down the lower side by their gravity. The slope is too flat for this idea to be entertained.

The littoral currents of the ocean also tend to modify the form of bars, as is seen in the Connecticut by the erosion of the southwestern edge of the bar, and in the form of the channels across the crest. These currents also carry off the finer portions of the sediment and deposit it in other localities.

About a mile to the southwest of the extreme point of Saybrook bar, a long sand shoal commences, and stretches westward for about six miles, averaging at the 18 feet depth about one-quarter of a mile in width. The material which forms this shoal probably comes from Saybrook bar, and is carried along by the incoming tidal current, while the outgoing tide keeps the channel open between it and the shore. The exact method of formation of this shoal, however, has not been investigated, and it may be due to other causes than those named. Its form and character are taken from the coast survey charts, and it cannot be seen to have any effect upon the form of the bar proper, although it is probably derived from it. On account of its distance, and the deep water between, it cannot properly be considered a part of the bar.

It is proposed during the coming season to commence some permanent works for improving the channel over the Saybrook bar. Some new ideas are believed to be advanced in the improvements contemplated, by taking advantage of the littoral currents to maintain the channel; but no description of them will be attempted at the present time. It is thought they will be successful, and, if so, they will be described at some future time.

LXXII.

ECONOMY OF RAILROAD CURVATURE.

A Paper by WILSON CROSBY, C. E., Member of the Society.

PRESENTED AT THE FIFTH ANNUAL CONVENTION, IN LOUISVILLE, KY., MAY 21ST AND 22D, 1873.

CHARLES HERMANY, C. E., IN THE CHAIR.

In the location of railroads, it frequently happens that a proposed curve joining two tangents may involve a large expense in construction, when, by shortening the arc between the same tangents, thus throwing the line on different ground, a great saving in the cost of preparing the road-bed may be effected. Whether or not this would be a measure of true economy, when the operation and maintenance of the sharper curve is considered, is a problem not so easily solved.

To answer this question satisfactorily, there should be known:

1st—The amount and kind of business the road will be required to do, and

2d—The relative cost of working and keeping in repair the track and machinery on the two curves in question.

The second of these considerations depends on the facility with which the vehicles traverse curves of different degrees of sharpness, a point upon which, unfortunately, but little accurate experimental knowledge exists, or at least has been accessible to the writer.

In the kindred problems of "equation for curvature" and "flattening of grades" to preserve a uniformity in the sum of the retarding forces, it is usual to consider the resistance arising from the curve, as increasing directly as the degree of curvature, which, although it may be nearly true for curves in ordinary use (not sharper than 6 or 8 degrees deflection), probably is not true for curves of 15 or 20 degrees deflection.*

* In the discussion of a paper on "Train Resistance on Railways," read before the Institution of Civil Engineers (England), January 24th, 1871, by Mr. W. Bridges Adams, Mr. G. J. Morrison gave formulæ for the partial resistance opposed by curves by a combination, of which the following expression for the total resistance due to the curve itself is obtained:

$$\frac{F\,W}{10\,R}\left(5\,(G+L)+F\,L\right)$$

in which F = coefficient of friction, W = weight of train, G = distance between centres of rails, L = length of wheel base, and R = radius of curve.

The Railroad Gazette of August 5th, 1871, contains an article taken from the American Railway Times, by the editor, Prof. G. L. Vose, from which is quoted, as follows: "The average of numerous experiments seems to show that the resistance upon a 10 degree curve at a speed of 20 miles per hour, is double that upon a straight line." The diminution of ascent in the grade to compensate for this resistance opposed by the curve, would be 0.46 per 100, or 24.29 feet per mile, corresponding to a decrease in traction of 9.23 pounds per ton of 2000 pounds.* This, according to the ordinary practice, would give 0.046 per 100 for each degree of curvature.

In commenting upon this paper, Mr. W. Atkinson cited Mr. B. H. Latrobe's rule (probably deduced from experiments on the Baltimore and Ohio R. R.), that on a curve of 100 feet radius the resistance was 30 pounds per ton, and inversely as the radius.

Mr. Chas. J. Quetil, Assistant Chief Engineer of the Texas and Pacific R.R., gives as this resistance—

$$\frac{F\,W}{2\,R}\sqrt{G^2+L^2}$$

Molesworth, in his "Pocket-Book," gives the resistance encountered by trains on a level straight track, in pounds, per ton of 2240 pounds, as

$$\frac{V^2}{171}+8$$

in which V is the speed in miles per hour. This is probably a formula derived from the experiments of Mr. D. K. Clark, as he also gives the same. Molesworth says, "The resistance of curves may be reckoned as one per cent. for each degree of the curve occupied by the train."

Mr. D. C. McCallum, General Superintendent of the New York and Erie R. R., in a report dated March 25th, 1856, on experiments made on that road the previous September, says: "After a careful examination and comparison of the loads moved upon the ruling grades and curves of the various sections of the road, it is assumed that the friction of the cars is $4\frac{1}{2}$ pounds per ton of 2000 pounds; the resistance of curves $\frac{1}{2}$ pound per ton per degree of curvature per 100 feet; and the additional friction of the engine $\frac{1}{2}$ pound per ton of load on a level and straight line, or its equivalent.

Combining the direct resistance of the curve with that part of the "additional friction of the engine" due to the curve, the total resistance resulting from the curve becomes $\frac{5}{8}$ pounds per ton of 2000 pounds for each degree of curvature per 100 feet. It thus appears that all of the above authorities, whether speaking from a theoretical standpoint or from the results of experiments, concur in thinking that the resistance of the curve increases directly as the degree of curvature or inversely as the radius, the other circumstances remaining constant.

*Since the resistance is to be doubled, that incident to the curve (the added part) will be equal to that upon a straight line at the given speed of 20 miles per hour, which according to Molesworth's formula is, for tons of 2000 pounds:

$$\frac{V^2}{191.52}+7.14 \text{ pounds.}$$

For a speed of 20 miles per hour or $V=20$, this becomes 9.23 pounds, and will therefore be the amount of resistance due to the curve itself. The traction due to a grade rising Y in 100, in pounds per ton of 2000 pounds is $2000 \div 100$, which by the hypothesis is equal to 9.23 pounds. Hence,

$$Y=\frac{100\times 9.23}{2000}=0.46$$

The resistance peculiar to the curve, as well as that of grade, is independent of the speed, although the whole amount experienced on the curve or grade is dependent on the speed as it would be on a straight and level line. The resistance on the straight level line at the given speed is to be added to the amount proper to the grade or curve, or both, to give the total to be overcome.

Mr. Charles Ellet, Chief Engineer of the Virginia Central R. R., in his report on the working of the "Mountain Top Track," says that with a locomotive, having 3 pairs of 42-inch drivers, coupled, but set in a flexible beam truck, and a wheel base of 9 feet and 4 inches, it was found that a curve of 300 feet radius, on a grade of 237.6 feet per mile, offered more resistance than a grade of 296 feet per mile on a tangent, showing that a flattening of 58.4 feet per mile was not sufficient to compensate for the resistance of the curve, since "the velocity with a constant supply of steam, was promptly retarded on passing from a straight line to a curve, and promptly accelerated again on passing from the curve to the straight line." When, however, a small quantity of oil was applied to the flange of the engine, it became "no longer possible to determine whether grades of 237.6 feet per mile on curves of 300 feet radius, or grades of 296 feet per mile on straight lines, were traversed most rapidly by the engine." Thus, with these conditions (flexible truck, small total wheel-base, and oiled flange), diminishing the grade 58.4 feet per mile, appeared to exactly compensate for the resistance arising from a curve of 300 feet radius, or a 19.1 degree curve. A flattening of 58.4 feet per mile for a 19.1 degree curve would be 0.0579 per 100 for each degree.

Molesworth's rule for finding the additional power required by curves, is to take it as one per centum of that on a straight line, for each degree of curve occupied by the train. In the deductions given by Prof. Vose, no mention is made of the length of the train, but it is given as "the average of numerous experiments." To make Molesworth's rule conform with this, would require a train 1,000 feet long, which, for an average, is rather long.*

This circumstance, taken in connection with Mr. Ellet's experience, would indicate, either that Molesworth's one per cent. increase was not large enough, or that the resistance increases more rapidly than the curve sharpens; that is, a ten-degree curve, for instance, would oppose, in 20 degrees of deflection, more resistance than 20 degrees of a one-degree curve; or it may show an error in both particulars.

In order to compare several curves, joining the same tangents, it is well to have a common standard to which to refer them. Let the chord of a one-degree curve, connecting the tangents, be taken for this pur-

* The trains employed by Mr. Ellet were probably not more than 120 to 150 feet in length. The experiments alluded to by Prof. Vose were principally those made on the Baltimore and Ohio R. R., and on the New York and Erie R. R., for an account of which, see appendix to his "Manual."

pose. If the traction and the depreciation of track and rolling stock on a curve were the same as those on a straight line of the same length, the different curves would be to each other, and to the chord, in value, inversely as their lengths; but, as has been seen, something must be added for the increased resistance and cost on the curves.

Let the following notation be adopteed:

R = radius of one-degree curve = 5729.65; r = radius of curve under consideration; a = angle of deflection of tangents (number of degrees in exterior angle); x = number of degrees subtended by the train, when the curve is long enough to contain the whole of it; c = amount of capital, the interest of which will perpetually operate and maintain one foot in length of straight line, at the required speed, together with the proportional part of the equipment; s = resistance encountered per foot run on a straight line; and i = the ratio of the increase in cost to the increase in power required by the curves.

If an increase of 100 per cent. in power on a curved portion of the road entails an additional expenditure of 25 per cent., in working and keeping in repair that part (including the machinery) over what would have been required had it been straight, $i = 25 \div 100 = \frac{1}{4}$. Multiplying the additional power by "i" gives the addition necessary to reduce the curve to a length of tangent equivalent in cost. The distance between the beginning and end of a one-degree curve, by the way of the chord joining those points, is represented by $2\ R\ \sin.\ \frac{a}{2}$ or, as measured around by the tangents and curve between the same points, by

$$2\ (R-r)\ \tan.\ \frac{a}{2} + \frac{2\ r \times 3.141593.a}{360} = 2\ (R-r)\ \tan.\frac{a}{2} + 100\ a\ \frac{r}{R}$$

very nearly. Taking merely the excess of length over the chord, this becomes

$$2\ (R-r)\ \tan.\ \frac{a}{2} - 2\ R\ \sin.\ \frac{a}{2} + 100\ a\ \frac{r}{R}.$$

Assuming that "x" increases uniformly to the full extent required by the train, whatever may be the value of "a," the expression

$$\left[2\ (R-r)\ \tan.\ \frac{a}{2} - 2\ R\ \sin.\ \frac{a}{2} + 100\ a\ \frac{r}{R}\left(1 + \frac{x}{100}\right)\right]s,$$

will give (by Molesworth's rule) the total excess of resistance to be overcome in moving from the place where the one-degree curve would begin to the place where it would end, by way of the tangents, and around the curve of radius, "r," over that experienced on a length of straight line equal to the chord of a one-degree curve. The excess of cost will be

$$2\ c\left[(R-r)\ \tan.\ \frac{a}{2} - R\ \sin.\ \frac{a}{2} + 50\ a\ \frac{r}{R}\left(1 + \frac{x\ i}{100}\right)\right] (A).$$

To make this readily available, the portion within the brackets should be tabulated. Then, to ascertain the relative economy of two curves, the difference between the tabular numbers corresponding to these curves for the given deflection of tangents is to be multiplied by twice the capitalized cost of the operation and maintenance of one foot of straight line; the result is the saving, in favor of the curve, of large radius. This, compared with the increased cost of construction on this curve, will show the net economy of the two lines. It is the difference between the tabular numbers that is wanted in all cases, and this difference will be independent of the length of the train; of the actual resistance due to the curve, whether one per cent. or more (provided that it varies directly as the sharpness); and of the increased cost of overcoming this resistance, since, in obtaining this difference, these cancel, the length of train remaining the same throughout the investigation.

Omitting the cost would be equivalent to striking $\frac{xi}{100}$ from (A). In a table made from the part of this formulæ within the brackets, with $\frac{xi}{100}$ omitted, the difference in tabular numbers would be half the difference in the length of the lwo lines compared. This, being the most useful table, is the one given.*

*In the construction of the table, the degree of curvature has been taken as exactly in the inverse ratio of the radius, which is not true. The error resulting from this slight inaccuracy (a triffling increase in the tabular numbers for the sharper curves) will be very small, and was thought to be of little moment in view of the uncertainty attending some portions of the theory, and of the labor saved by this means, as compared with the use of the exact formula.

Example in use of the table—On a proposed line of railroad there are two tangents which make an exterior or deflection angle of 100° with each other. Between them there is a rocky hill of such shape and situation, that the difference in the cost of preparing the road-bed to connect them by a 2° or a 6° curve (the maximum degree of curvature permissible on the road) will be $30,000. It is required to find which of the two curves is the most desirable, and by how much. The business of the road is expected to be such that the value of one foot in length saved will be, in the items of cost of track, fencing, proportion of equipment and capitalized cost of operation and maintenance, equal to $9.

Tabular number for 6° curve for 100° deflection, is	2134.45
" " 2° " "	1525.01
Difference in tabular numbers or half difference in distance =	609.44
Total distance saved by line of 2° curve = 1218.88 feet, or say	1219.00
Additional cost of graduation on line of 2° curve =	$30,000
Capitalized saving on line of 2° curve is 1219 × 9 =	10,971
Difference or net saving by line of 6° curve =	$19,029

For the same tangents and hill, but for an anticipated traffic that would make the capitalized worth of one foot in distance equal to $30, we should have:—

Capitalized saving on line of 2° curve is 1219 × 30 =	$36,570
Additional cost of graduation on line of 2° curve =	30,000
Net saving by line of 2° curve =	$6,570

With regard to the assumptions made, the following remarks are applicable :

First.—As to Molesworth's uniform addition of one per cent. This supposes that up to the maximum degree of curvature employed, the resistance increases inversely as the radius, a hypothesis which some of the experiments do not seem to confirm for curves of small radii. Inasmuch, however, as the cost to be used as that pertaining to a straight line, will generally be derived from experience on the whole length of railroads, containing both curves and tangents, and will therefore be too large for straight lines alone, adding one per cent. of it is actually adding more than one per cent. of the cost of operating and maintaining a given length of straight line : and this excess over the nominal percentage required by the rule increases with the sharpness of the curve, where, as has been seen, the resistance per degree is greatest. Here, then, is a compensation of errors more or less complete.

Second.—As to the increase in the value of "x" after the length of the train reaches beyond the limits of the curve ; it is still increased, for the reasons stated below :

1st. Because this would be a mode of showing an increased resistance in the sharper curves, where it is most apparent in practice.

2d. Because the traction on the curve probably does increase somewhat with the length of the train, even when the latter extends beyond the curve—the greater length on the tangents, aside from the weight, causing it to draw harder around the arc.

3d. Because if "a" is fixed as the superior limit of "x," the addition to the cost due to the curve resistance, derived from the expression $50\ a \frac{r}{R} \frac{xi}{100} 2\ c$, might be less for a long train and sharp curve (not long enough to contain it) than for the same train and a flatter curve—which is absurd and contrary to the fundamental hypothesis.

4th. Because the equation thus becomes much simplified, and the state of knowledge on the subject is not such as to warrant the conclusion that exact results could be obtained, even with a very complicated formula.

Granting the foregoing premises, it appears that the relative cost of the operation and maintenance of the connection between two given tangents is directly as the length of track between common points, without farther consideration as to the sharpness of either arc, provided it is not so sharp as to be beyond the limit of economical working.

TABLE.

Degree of Curve.	Deflection of Tangents or Value of "a."													Degree of Curve.
	10°	20°	30°	40°	50°	60°	70°	80°	90°	100°	110°	120°	130°	
1	0.60	5.05	17.05	40.35	78.54	135.18	213.59	317.04	448.51	610.86	806.56	1037.95	1307.16	**1**
2	1.25	10.20	34.68	83.06	164.43	289.18	469.57	720.91	1163.33	1525.01	2147.96	2999.97	4200.81	**2**
3	1.46	11.92	40.56	97.29	193.06	340.52	554.90	855.54	1268.27	1829.73	2595.09	3553.98	5165.35	**3**
4	1.57	12.78	43.50	104.41	207.38	366.19	597.56	922.85	1370.75	1982.09	2818.66	3980.98	5617.63	**4**
5	1.63	13.29	45.26	108.68	215.97	381.59	623.15	963.24	1432.23	2073.51	2952.80	4177.18	5937.00	**5**
6	1.68	13.64	46.44	111.53	221.70	391.85	640.22	990.16	1473.22	2134.45	3042.23	4307.99	6129.91	**6**
7	1.71	13.88	47.27	113.56	225.79	399.19	652.41	1009.39	1502.49	2177.98	3106.10	4401.41	6267.70	**7**
8	1.73	14.07	47.90	115.09	228.85	404.69	661.55	1023.82	1524.45	2210.63	3154.01	4471.49	6371.04	**8**
9	1.75	14.21	48.39	116.26	231.24	408.96	668.66	1035.04	1541.53	2236.02	3191.27	4525.99	6451.42	**9**
10	1.76	14.32	48.79	117.22	233.15	412.39	674.35	1044.01	1555.19	2256.34	3221.08	4569.59	6515.72	**10**
11	1.77	14.42	49.11	118.00	234.71	415.19	679.00	1051.36	1566.37	2272.96	3245.47	4605.26	6568.33	**11**
12	1.78	14.50	49.37	118.65	236.01	417.52	682.88	1057.47	1575.68	2286.81	3265.79	4634.99	6612.18	**12**
14	1.80	14.62	49.80	119.67	238.06	421.19	688.98	1067.10	1590.33	2308.58	3297.74	4681.71	6681.03	**14**
16	1.81	14.71	50.11	120.43	239.59	423.94	693.55	1074.30	1601.30	2324.90	3321.68	4716.74	6732.75	**16**
18	1.82	14.78	50.35	121.02	240.79	426.08	697.10	1079.91	1609.85	2337.60	3340.32	4743.99	6772.94	**18**

LXXIII.

AN ACCOUNT OF THE ERECTION OF A DRAW-BRIDGE WITHOUT FALSE WORKS.

A Paper by C. S. Maurice, C. E., Member of the Society.

Read at the Evening Meeting, December 17, 1873.

In giving the following I would say, by way of preface, that though the matter may not be of sufficient importance to be made the subject of a formal communication to the Society, or to be embodied in its Transactions, yet the fact that the plan of raising was hastily improvised with only the crudest materials at hand for its execution, may serve as some apology for occupying a portion of your time this evening.

The structure to which I refer, is the iron bridge built for the Alabama Central R. R. over the Tombigbee River, some six miles below the town of Demopolis. The width of the river here is from 500 to 600 feet between its banks, and the depth of water varies from 6 to 8 feet in dry seasons to 60 or 65 feet at the time of heavy freshets. The first railroad bridge at this point was completed in 1866, and consisted of 2 Howe truss spans of 200 feet each, and a pivot span of the same dimensions, the whole structure being supported on timber piers. In the substructure of the new bridge, which was built on the site of the old one, brick piers with stone copings were used, and the superstructure consisted of 2 fixed spans of 160 feet each, with a pivot span of 260 feet, all of wrought-iron.

The construction of the new piers was begun in the summer of 1872, by Col. M. B. Prichard, of Montgomery, Ala., and completed about December 1st following. The contractors for the superstructure, who also removed the old bridge and furnished false work of sufficient strength to carry the trains while the work was in progress, began operations late in October, 1872. The fixed spans of the old bridge were taken down and the iron spans erected in their place in the usual manner. This occupied till the first week in December, and the river being then at its extreme low-water mark, and with every prospect of continuing so for at least a month to come, the trestles were erected under the old draw for its removal, and by the middle of the month the contractors were ready to

begin the erection of the new pivot span.* A heavy rain, however, having set in a few days before, it was deemed advisable to see what its effects would be on the river before trusting the iron upon the false work. The storm continued without interruption for an entire week, and the river rose with great rapidity to a height of 40 feet, bringing down immense quantities of drift-wood.

The trestles were, of course, carried out, and as there was not the slightest probability of the water falling, so as to admit of their being replaced before the following summer, all idea of completing the bridge in the ordinary manner had to be abandoned.† The following plan of erecting by the use of temporary trusses was then determined on. Between the pivot and end supporting pier on the west side, and at a distance from the latter of about 25 feet, there was left standing one of the old timber piers of the original bridge, and this being readily accessible from the fixed span by ordinary stringers, the opening which had to be spanned to reach the pivot pier was reduced to 95 feet. Two light timber trusses, of the Pratt system, 98 feet in length, were then framed and erected on the river bank, and each one launched separately and floated into position between the pivot pier and timber pier just mentioned.

In the meantime the river had fallen considerably, so that the height from the surface of the water to the top of the piers was about 45 feet; the trusses were raised from the water by means of tackles and gears and two stout gin poles, and secured in the position they were to occupy. The west half of the draw and the gallows frame were then erected without difficulty. On the east side, however, there was nothing that could be used as a support for the temporary trusses between the new piers, and the distance between them being 114 feet in the clear, the trusses were about 20 feet too short to reach across. To reduce the opening so as to admit of using the same trusses, a heavy timber platform, 20 feet in length, was thrown out from the pivot pier, the outer end being firmly supported by suspension rods from the top of the gallows frame. The timber trusses were then hung from the upper chords of the completed iron trusses of the west section, and moved forward over the pivot pier until one end projected some 20 feet beyond the timber platform.

* It might be mentioned that during the season of low water there was no occasion for opening the draw, as the largest boats on the river could readily pass under the fixed spans without lowering their smoke stacks.

† It may be asked why the draw was not erected in its open position, at right angles to the line of the bridge. The reason for not doing so was, that no "protector" had been built, and as the nature of the river bed almost precluded the driving of piles, the cost of erecting trestle work capable of resisting the force of the current and the drift, during the rise of the river, would have been so great as to render this plan objectionable on account of the expense.

It was in this transfer of the trusses that the principal difficulty was encountered, and the want of wire rope and other suitable appliances for handling heavy weights was most seriously felt. The trusses when detached from each other were, of course, exceedingly limber, and great care was necessary in moving them to prevent injury by buckling.

A Manila cable, 4 inches in diameter, was stretched from the top of the gallows frame to the top of the fixed span on the opposite side, and on this it was designed to suspend the trusses, one at a time, while moving them into their final position ; the height of the cable above the top of the piers and its angle of inclination being such, that it was supposed that the trusses, when hung upon it, would move readily under the influence of gravity. Unfortunately, however, the cable had been thoroughly wet by a heavy rain, and though the stretch was thought to be pretty well taken out by tackles, the effect of the soaking proved much more serious than was expected. As the weight of the truss came upon the cable it gradually stretched and sagged down some 70 feet, so that the truss, instead of being held above the level of the copings, was about 30 feet below them.

The plan of transferring by cable having thus proved a failure, the gin poles and tackles were again resorted to, and by means of them the trusses were finally drawn up to their place, one end resting on the east end pier and the other on the timber platform above described. As soon as the timber trusses were connected by putting in the floor beams and laterals, the erection of the iron trusses of the east section of the draw was begun, and in 2 days—20 working hours—from that time the bridge was completed and the draw opened for the passage of boats.*

The expense of erecting in the manner described proved rather less than was anticipated. The materials for the temporary trusses cost about $200, and the entire work of raising the pivot span, including the framing, raising and transferring of the timber trusses was accomplished by a force of 18 men in 24 working days, so that the entire cost of erection did not much exceed $1,300.

* During the erection of the east section of the draw, the river being so high that boats could not pass under the bridge, navigation at this point was pretty effectually closed, and in anticipation of this event, on complaint of the steamboat owners of Mobile, notice had been served by the United States authorities, forbidding any prosecution of the work that would prevent the free passage of boats. To avoid the annoyance of a number of suits at law, a compromise was effected between the contractors and the several steamboat companies, by which the former agreed to pay a fixed rate per day for every day that navigation should be obstructed. The claims for damages on this account, amounting to $3,000, were paid by the contractors on the completion of the bridge.

Fig. 1.

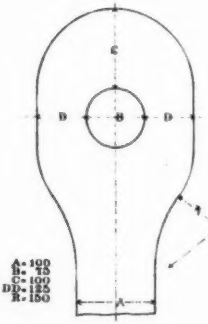


Fig. 2.

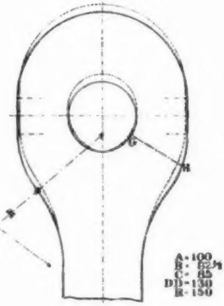


Fig. 3.

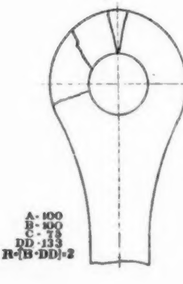
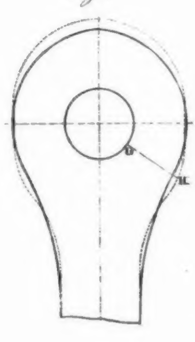


Fig. 4



SECTION THROUGH CENTRE OF HOLE.

Fig. 5.

SECTION THROUGH CENTRE OF HOLE.

Fig. 8.

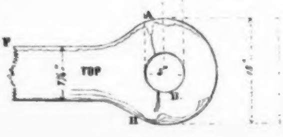
VIEW OF EDGE A F.

Fig. 6.

VIEW OF EDGE E L

Fig. 9

Fig. 7.



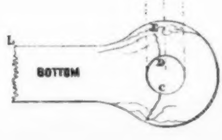


Fig. 10.

LXXIII.

PROPORTIONS OF THE HEADS OF EYE-BARS.

A paper by CHARLES MACDONALD, C. E., Member of the Society.

READ AT THE EVENING MEETING, JANUARY 21ST, 1874.

In the discussion of a paper on the proportion of pins, read by Mr. Bender, before the Society during the past year,* the importance of a properly proportioned eye-bar was referred to, as exercising a considerable influence on the size of the pin. Reference was had to the published account of experiments made in England, up to the year 1869, from which it appeared that in order to secure the full strength of a bar it is necessary to proportion the head according to the dimensions given in Fig. 1. The method observed in the manufacture of these heads is not stated in the published account of the experiments, but it is presumed that the bar is first rolled to the full width of the head, and then drawn down between the heads in a reversible mill to the required width, leaving the heads to be forged to the proper shape under a hammer. The results of the following experiments confirm the general accuracy of the English standard, and an examination of the change in form of the head under strain will be of interest in assigning reasons for the conclusions arrived at.

The tests were made at the works of the Watson Manufacturing Company of Paterson, N. J., during the month of December last, under the direction of Mr. O. Chanute, for the purpose of determining the character of the iron used in links for the new iron bridges on the Erie Railway. The testing machine was a hydraulic press of approved construction, and the behavior of the iron was believed to be in the main satisfactory. The results in this particular will not be reported in detail, further than relates to the subject under consideration. The heads of all the bars tested were made at the works of the Phillipsburg Manufacturing Company, by the process known as die forging. The end of the bar is slightly thickened and drawn down to a wedge shape; a pile of scrap is then placed upon it, and the whole heated to a welding heat, after which it is drawn out under a steam hammer, and forged into the

* "Proportion of Pins used in Bridges," by Charles Bender, C. E., read before the Society, April 2d, 1873, and afterwards published in an extended form.

proper contour of the head by means of a vertical die, half of which is cut out of the anvil and half out of the hammer. Three bars having a section of 4 by $\frac{7}{8}$ inches and 6 feet long, were broken in the body of the bar, under an average strain of 54,400 pounds per square inch; all of the bars indicating the same condition of fracture as the specimen submitted. In each of these, the heads were proportioned as in Fig. 2. After rupture the heads were found to have assumed the form indicated by the dotted lines; from which it would appear that the proper disposition of material upon a line G H is of the first importance, as tending to transfer strain from the bar to the back of the pin without undue concentration at the edge of the pin-hole. If the amount of material upon this line were sufficient to prevent change of form, the lines of strain from the bar will arrive at the section DD in a direction parallel to the bar, and the area of this section would then not require to exceed that of the bar itself. In practice it is not possible to effect this result absolutely, as will be noticed in the movement in this particular head, hence it is proper to increase that section by a certain proportion of the bar section; in this case it is 30 per cent., while by the English standard it is 25 per cent. In determining a proper depth behind the pin it should be borne in mind that from the nature of the manufacture of a head the fibre of the iron cannot be disposed in the direction of the strain with the same uniformity on this line as elsewhere, hence the necessity of allowing a more liberal margin for safety.

At the same time and place two bars were tested, having heads proportioned as in Fig. 3. No. 1 burst at the crown, under a strain of 44,000 pounds per square inch, the fracture showing burnt iron for a distance of $\frac{3}{4}$ inch inwards from the point. The broken head was subsequently removed, and a new one made from Passaic Rolling Mill iron, was welded to the bar by Watson's hollow fire process. Upon application of the strain the bar broke through the Passaic iron with 51,600 pounds per square inch. No. 2 burst at the side of the head, on two lines, under a strain of 50,000 pounds, the fracture showing slightly burnt iron. It is to be regretted that the scantling of these last bars was not the same as in the previous cases, in order that the comparative effect of pin diameter might have been eliminated. If, however, a head be designed for a 4-inch bar upon the same basis as in Fig. 3, it will be found (see Fig. 4) that the section on a line G H is considerably less than in the standard before you, and the depth behind the pin is also deficient.

The question as to whether the dimensions assumed in Fig. 2, are the

correct ones for heads manufactured by forging or welding can scarcely be determined from the limited number of experiments referred to; but by comparing the results with those obtained upon the same subject abroad, it may be assumed that they approach very nearly to an accurate standard. One fact seems to be clearly indicated, namely, that it is not by increasing the section DD that stability is to be obtained, but by thickening the head in front of the pin in order to secure a proper distribution of strain in DD. Whether this rule applies to heads formed by the upsetting process remains to be proved. The present practice in some establishments is to make DD 50 per cent. greater than the bar; probably to counterbalance the effect produced by distortion of fibre in the manufacture; and because of the difficulty of forcing the metal far enough back to maintain a proper width in front of the pin.

Inasmuch as the diameter of pin must first be known before the head can be proportioned, a table of pin diameters is annexed, varying for widths between 2 and 7 inches in flat iron, and up to $4\frac{3}{8}$ inches for squares and rounds. This table has been calculated upon the supposition that for flats thinner than $3\frac{1}{2}$ to 1, the pin diameter should be 75 per cent. of the width of the bar, as indicated by the English standard. But inasmuch as the experiments upon which this ratio was determined, were made with the pin supported on each side of the eye;* it becomes necessary to consider the effect of a thick bar upon a pin projecting as from the top chord casting of a bridge. For all practical purposes we may assume the pin to be in the condition of a cylindrical beam fixed in position at its supports and loaded with a weight equal to the strain upon the bar, distributed uniformly over a length of pin equal to the thickness of the eye. The formulæ expressing the diameter of pin which shall not be subjected to a greater strain upon its extreme fibres than 10,000 pounds per square inch, under the above circumstances, are as follows:

For flat bars—

$$D = 1.721\, t \sqrt[3]{n}, \qquad \text{(Eq. 1.)}$$

In which t = thickness, and t n = width. For square bars, n = 1. For round bars, in which the thickness of head is $\frac{1}{8}$ of an inch less than the diameter, of bar, the expression becomes

$$D = \sqrt[3]{4\, d^3 - \tfrac{1}{2}\, d^2} \qquad \text{(Eq. 2.)}$$

In which D = diameter of pin, and d = diameter of bar.

* The diameter of pin in the experimental bar was determined with reference to heavier bars in the structure for which it was intended.

DIAMETER OF PINS.

FOR FLAT BARS.												FOR ROUND AND SQUARE BARS.					
Thickness in Inches.	WIDTH OF BAR IN INCHES.											Diam. or side of Square.	For Round Bars.	For Square Bars.	Diam. or side of Square.	For Round Bars.	For Square Bars.
	2	2 1/2	3	3 1/2	4	4 1/2	5	5 1/2	6	6 1/2	7						
1/2	1 1/2	1 7/8	2 1/4	2 5/8	3	3 3/8	3 3/4	4 1/8	4 1/2	4 7/8	5 1/4	1	1 5/16	1 3/4	2 3/4	4 5/16	4 3/4
5/8	1 5/8	1 7/8	2 1/4	2 5/8	3	3 3/8	3 3/4	4 1/8	4 1/2	4 7/8	5 1/4	1 1/8	1 3/4	1 15/16	2 7/8	4 1/2	5
3/4	1 7/8	2	2 1/4	2 5/8	3	3 3/8	3 3/4	1 1/8	4 1/2	4 7/8	5 1/4	1 1/4	1 15/16	2 3/16	3	4 11/16	5 3/16
7/8	2	2 3/16	2 5/16	2 5/8	3	3 3/8	3 3/4	4 1/8	4 1/2	4 7/8	5 1/4	1 3/8	2 1/8	2 3/8	3 1/8	4 15/16	5 7/16
1	2 3/16	2 3/8	2 1/2	2 5/8	3	3 3/8	3 3/4	4 1/8	4 1/2	4 7/8	5 1/4	1 1/2	2 5/16	2 5/8	3 1/4	5 1/8	5 5/8
1 1/8	2 3/8	2 9/16	2 3/4	2 7/8	3	3 3/8	3 3/4	4 1/8	4 1/2	4 7/8	5 1/4	1 5/8	2 9/16	2 13/16	3 3/8	5 5/16	5 13/16
1 1/4	2 1/2	2 11/16	2 15/16	3 1/16	3 1/4	3 3/8	3 3/4	4 1/8	4 1/2	4 7/8	5 1/4	1 3/4	2 3/4	3 1/16	3 1/2	5 1/2	6 1/16
1 3/8	2 11/16	2 15/16	3 1/8	3 5/16	3 7/16	3 9/16	3 3/4	4 1/8	4 1/2	4 7/8	5 1/4	1 7/8	2 15/16	3 1/4	3 5/8	5 5/8	6 1/4
1 1/2	2 7/8	3 1/16	3 1/4	3 7/16	3 5/8	3 3/4	3 15/16	4 1/8	4 1/2	4 7/8	5 1/4	2	3 1/8	3 1/2	3 3/4	5 7/8	6 1/2
1 5/8	3	3 1/4	3 7/16	3 5/8	3 13/16	4	4 1/8	4 5/16	4 1/2	4 7/8	5 1/4	2 1/8	3 5/16	3 11/16	3 7/8	6 1/8	6 11/16
1 3/4	3 3/16	3 3/8	3 9/16	3 13/16	4	4 3/16	4 3/8	4 1/2	4 5/8	4 7/8	5 1/4	2 1/4	3 1/2	3 7/8	4	6 5/16	6 7/8
1 7/8	3 3/8	3 9/16	3 13/16	4	4 3/16	4 3/8	4 9/16	4 11/16	4 7/8	5	5 1/4	2 3/8	3 11/16	4 1/8	4 1/8	6 1/2	7 1/8
2	3 1/2	3 3/4	3 15/16	4 3/16	4 3/8	4 9/16	4 11/16	4 7/8	5	5 1/8	5 1/4	2 1/2	3 15/16	4 5/16	4 1/4	6 11/16	7 5/16
2 1/8	3 11/16	3 7/8	4 1/8	4 5/16	4 9/16	4 11/16	4 15/16	5 1/16	5 3/16	5 5/16	5 1/2	2 5/8	4 1/8	4 9/16	4 3/8	6 7/8	7 9/16

By solving Eq. 1 for values of n greater than 3½, it will be found that the value of D obtained will be less than 75 per cent. of the width of the bar; hence, for all widths above this limit, the ratio 75 must be taken as determining the pin diameter. For other widths the formulæ as above have been used in calculating the table.

Mr. Collingwood—Mr. Macdonald has pointed out two primary considerations in the economical use of iron—form and condition of the metal. Unless these are thoroughly cared for, the consequent result will be an increase of weight in our structures. Admitting, however, that these are as required, it seems to me that a third element should be taken into account (even in deciding upon the first) and that is, the methods pursued in forming eyes, bolts and the like—not, however, referring to the state in which the process employed leaves the iron, chemically considered (that is burnt or unburnt), but to its physical condition, its compactness and the uniformity of its fibre.

In preparing the lower anchor bars of the East River Bridge for insertion into the masonry, four bars were left in the acid a little too long, and the result was to show the fibre in the eyes very plainly. The bars were 3×7 inches section, about 13 feet long from centre to centre of pin-holes, which were 5 inches, and the eyes 15 inches in diameter. The eyes were formed by hydraulic pressure, being first upset on the end and then pressed on the flat into a die which gave the perfect shape. The two sides, edges and section of the iron in the pin-hole of one of the eyes are shown in the figures. The result of this process, as will be at once seen, Figs. 5 to 10, is to cause the fibres to fold back on themselves, and leave on each face (almost directly in the position pointed out by Mr. Macdonald as needing greatest strength) lines more or less depressed, the average depth being $\frac{1}{16}$ to $\frac{1}{32}$ inch. The lines on the opposite faces were never opposite, but were from, on one side ¾ to 1½ inches further from the centre than on the other; in the bar shown, this distance was ¾ inch. In some of the eyes also there was an appearance of looseness about the pin-holes shown by an actual separation of the fibre for ⅛ inch. The bars were tested to 20,000 pounds per square inch without set. It is nevertheless certain that the iron in the eye was not in condition to give its greatest strength. Even if we decline to recognize the existence of fibre, we cannot deny that iron is stronger in the direction in which it is rolled than in any other, and that well-worked, compact iron is stronger than that which is slightly worked.

I have been asked to describe the process of preparing the bars

for the anchor chains of the East River Bridge, before they were covered up in the masonry. The bars had been painted, and when delivered had considerable rust upon them. A long shed was prepared with an overhead traveling truck (cheaply made), to which were attached two differential pulley blocks for lifting the bars. Underneath, at a convenient height for the workmen, was a double line of rails on which the bars could be slid along for painting; at one end of the shed were placed, side by side, five vats. In the first was a solution of potash to remove grease and paint; in the second and fourth, water for rinsing; in the third, dilute sulphuric acid, and in the last was lime water; the potash and lime vats were heated by steam. The strength of the solution is not very material, as it only affects the time required to produce the desired result.

Four bars were usually in each vat at once. The intent was to remove the scale entirely by the acid, but we soon found that the bars were eaten too much before it came off. Resort was then had to hammering to detach it, the object being to secure a clean metallic surface. After cleansing, the bars were left in the boiling lime-water until thoroughly heated. They were then rinsed quickly, and while hot, coated with raw linseed oil. After this had hardened thoroughly, raw linseed oil mixed with Spanish brown and then boiled oil with Spanish brown were applied. The pin-holes were then thoroughly cleaned, made smooth and rubbed with raw oil.

After the bars were put in position in the masonry, they were subjected to a heavy upward strain to bring them to bearing; they were then adjusted and wedged; next, thin grout was poured around the eyes and pins and rich concrete filled around the bars. Recent examinations at Niagara show that with our American cements, this process affords an absolute protection against rust.

ERRATA.—In "LXIX, Tables of the Strength of Cast Iron Columns," on page 297, second line from the bottom, for $10^{2.55}$, read $10^{3.55}$; on page 299, tenth line from the bottom, for $\frac{l}{d} - 1$, read $\frac{l}{d} = 1$; and on page 303, where $\frac{l}{d} = 38$ and $r = 4$, for 15565 read 13565.

LXXIV.

EXPERIMENTS ON THE TENSILE STRENGTH OF BAR-IRON AND BOILER-PLATE.

A Paper by C. B. Richards, M. E., Member of the Society.

Read at the Evening Meeting, January 21st, 1874.

The object of presenting this paper is to communicate the particulars and results of a number of experiments made with the Colt's Armory testing machine on the tensile strength of several kinds of American bar-iron and boiler-plate; also to describe a gauge, designed by the writer, for measuring the elongation and contraction of specimens under varying strains. Tabulated results of the experiments are given, and a drawing is added, which shows the forms of the specimens used, and also shows the gauge with its fastenings.

Some of the experiments were made for the purpose of ascertaining to what extent the specimen's shape modified the results given by the tests, and a description of these should properly be prefaced by some allusion to similar investigations by Kirkaldy, who has quite fully described several experiments made by him to determine the influence which the shape of the specimen had on results obtained from tests of the tensile strength of different English irons. Kirkaldy found that the influence of the specimen's shape on the results varied with the softness or ductility of the material, and that this influence was important in the case of soft or ductile irons, but became hardly appreciable with hard and brittle materials. We may, therefore, conclude that we cannot compare tests of different irons, nor even compare different tests of the same kind of iron, if the several tests were made with specimens unlike in general form, without determining, by experiment, what effect the variations in the forms of the specimens had in modifying the results for the particular irons in question.

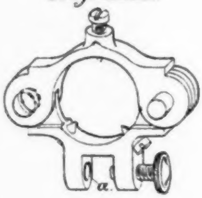
On account of discrepancies between the results obtained by the writer and those arrived at by others, from tests of the tensile strength of some American irons, it became important to ascertain, by direct experiment, what the difference would be between the values of strength

given, for these particular irons, by specimens shaped like those formerly used in the United States government experiments, and those given by specimens of the form adopted by Kirkaldy. In order to do this fairly, a number of specimens of each kind were broken, which had been prepared as follows: A bar of the iron was cut into pieces of suitable length for specimens, and the pieces, taken in the order in which they lay in the bar, were shaped, alternately, like the drawings *S* and *L*; *S* being made by turning, in the body of the piece, a narrow groove with a rounded bottom, by which the section was reduced to the proper diameter to insure fracture at that place, and *L* being formed by reducing a considerable portion of the central part of the piece to the same diameter as in *S*, making a slender, cylindrical part, 4 or 5 inches long. In the case of plate-iron, the specimens were made of the forms shown in the drawings at *A* and *B*; the reduced width being, in one case, made by cutting a narrow notch in the middle of each edge of the piece, and in the other case, by cutting away a considerable portion of the material from each edge, so as to leave the weakened part with paralleled edges 3 or 4 inches long. In what immediately follows, and in the tables, specimens shaped like *A* or *S* are called "short" specimens, and those like *B* or *L*, "long."

Tables I, II and IV give the particulars of these experiments in the cases of "Burden's Best" bar iron and "Bay State" boiler-plate. These tables show that, for "Burden's Best" bar, the "short" specimens gave 62,000 lbs. as the value for the tensile strength per square inch of the original cross-section of the finished specimen, while the "long" specimens gave only 49,600 lbs. for that value; the difference in the results corresponding to 25 per cent. of the smaller value. For "Bay State" plate, the "short" specimens gave 52,100 lbs., and the "long," 47,450 lbs. as the values for strength, the variation being 10 per cent.

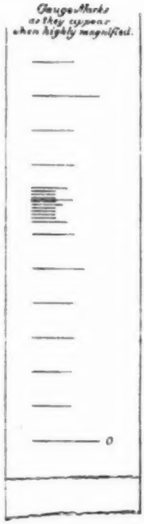
It will, however, be seen that the breaking strains, when referred to the areas of the fractured ends of the fragments of the specimens, give nearly equal values in tests of the same material, made with either kind of specimens. These values Kirkaldy calls "the breaking weight per square inch of fractured area," and they would, at first, seem to afford a means of comparing tests of different meterials, without taking into account the specimen's form. A moment's reflection will show, however, that, in the case of a "short" specimen, whose shape prevents any considerable reduction of the area of the cross-section in breaking, however ductile the material, the contraction of the cross-section may also be

Gauge Holder.

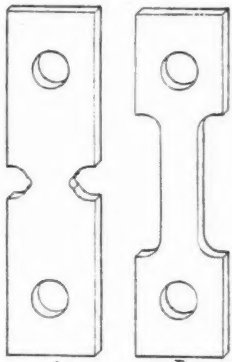


a.

Gauge.

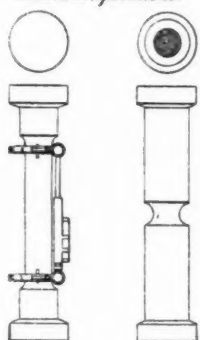


Boiler Plate Specimens



A. B.

Bar Iron Specimens.



L. S.

limited by the want of softness, or the brittleness of the material, and without knowing the specimen's form, and the extent of the influence of that form on the results, we cannot determine which circumstance produced the effect.

On considering the shape in which iron is usually found in structures where its office is to resist tensile strains, it becomes evident that tests made with "short" specimens are of comparatively small practical value. In order, then, to obtain much useful information about the qualities of irons from tensile tests, it is necessary to use a specimen of considerable length, that is to say, one in which the weakened portion is a somewhat slender cylinder, and not simply a short neck. From observing the forms of fractured specimens, it would seem probable that, in general, the length of the cylindrical weakened portion should be made at least equal to twice its diameter, and we may assume this to be a suitable minimum length.

While, then, it is desirable to avoid the use of the necked or "short" specimens, and this may therefore be dismissed from further consideration, it is not practicable to always use specimens of equal, or even approximately equal lengths; and if we are to compare the qualities of different materials by comparing tests made with specimens of various lengths, we must do so by a comparison of those effects of the tensile strains which are due to the qualities of the material, and which are not essentially influenced by the specimen's length. In this connection, one of the questions which arises relates to the best means of comparing the ductile qualities of the different materials; and it is here that the consideration of the reduction of the area of cross-section of a specimen, by stretching, or breaking, becomes of great value; for it is found that after we pass a certain limit in the length of the specimen, the contraction of the cross-section by the fracturing is not influenced by additional length.

Our choice in the means of measuring ductility seems to be limited to the consideration, either of this reduction of cross section, or else of the elongation which the specimen undergoes in breaking. Now the ratio of the ultimate elongation of a specimen to its original length, under some circumstances, varies with the length to a considerable extent; for example, this ratio, in a specimen of soft iron of 1 inch diameter and 2 inches length, would be much greater than in one 3 inches long; and in the latter the ratio would be greater than in a 6-inch specimen. The reason of this is, that ductile materials in breaking are

drawn out more in the immediate vicinity of the place of fracture than elsewhere. This excessive drawing out occurs just as the specimen is breaking, and after it has ceased to stretch in its other parts; and, for the same material, the extent of this final drawing out is nearly equal in specimens of different lengths, provided the specimen is not too short to permit this to take place without restraint. The length of the portion which is drawn out in this way varies with different materials, and also varies slightly with the diameter of the specimen, but it seldom exceeds twice the original diameter of the specimen, and extends usually for a distance of not more than one diameter from the fractured end of each fragment. As the length of the excessively drawn out portion is a constant quantity in specimens of various lengths, it is evident that the proportion of the length of this part to the whole length will be greater in the shorter than in the longer specimens, and the relative total elongation will therefore be greater in the former than in the latter.

This circumstance was illustrated in an interesting way by specimen No. 333 of "Burden's Best" iron. A spiral line, of one-fourth of an inch pitch, was traced on the cylindrical part of this specimen before it was strained. The whole length thus marked was originally 5.25 inches. After fracture, the pitch of this spiral line, although much increased by the stretching, was found to be uniform except in the neighborhood of the place of fracture, where the pitch ceased to be uniform and increased rapidly from a short distance each way from the place of fracture up to the fractured edges, at which place it was greatest. The specimen parted at about an inch from one of the shoulders. On each side of the place of fracture there were only three coils of the spiral line whose pitch was greater than the pitch at other places; there was, therefore, a space of one-fourth of an inch from the limit of the drawn out part to the shoulder, so that we may assume that, if the cylindrical part of the specimen had been only 1.5 inches long, the drawing out in fracturing would have been the same as if more room had been given, and the contraction of the area of cross-section of the fractured ends would also have been the same. Let us suppose, then, that the specimen had consisted of only that portion which was drawn out in the act of breaking. Its original length would have been 1.5 inches, and we find by measurement of the fractured specimen, that this particular part was increased in length to 2.1 inches, showing an elongation of 40 per cent. But if we measure the whole length of the broken specimen, from one

end of the spiral line to the other, we find that this distance, which was originally 5.25 inches, was increased by stretching and fracture to 6.7 inches, which corresponds to an elongation of about 28 per cent. only. The reduction of the area of the fractured ends would have been the same in both cases. In these cases, then, the value for ductility, if obtained from the contraction of area in breaking, would have been the same for both, but if obtained from the elongation the variation would have been 12 per cent.

We must, however, in many cases depend on measurements of the elongation of the specimen rather than on the reduction of cross-section. In testing high steels, for example, it is very seldom that a full contraction in the act of breaking can be obtained, for the brittleness of the material is so great, that just as the breaking strain is reached, very slight jars will occasion abrupt fracture without the final contraction taking place which is due to the act of breaking. Occasionally, however, very beautiful contractions of section can be obtained from high steels.

This subject has been considered at length, because it illustrates the importance of observing and recording all the particulars which relate to experiments on tensile strains.

A peculiar gauge was used for measuring the extensions of the specimens within the limits of their elasticity. This, together with one of the clamps or gauge-holders by which the gauge is attached to the specimens, is shown in the drawing. The clamp consists of two nearly semicircular steel straps, both ends of which interlock and are formed into hinge-like joints, whose pins are removable, so that the clamp may be opened and be applied to the specimen, which it will then encircle. When thus applied, the ends of the semicircles can be locked together by replacing the pins in the hinges. Three pointed screws, all lying in the same place and pointing radially inward, pass through the circumference of the clamp and serve to hold it immovably on the specimen. At the front of the clamp is an open jaw, shown in the drawing at *a*. One side of this jaw is recessed in a suitable manner to form a socket for a ball which is at one extremity of the gauge, and the other side is provided with a screw by which the ball may be pressed firmly into the socket and be held there without play. The clamps are fastened to the cylindrical part of the specimen, one near each end, with their jaws in the same plane with the axis of the specimen.

In the drawing of the specimen *L*, the clamps and the gauge are

shown *in situ.* The gauge consists of two scales or sets of lines, ruled on separate flat pieces of glass, which lie upon each other face to face, the faces on which the scales are ruled being in contact with each other, so that the two scales are in the same plane. On one of the scales, a distance of an inch is divided by fine lines into one hundred parts, and in the other scale there are ten lines the one-thousandth of an inch apart. The two glasses are fastened in separate steel frames, which are fitted together so as to slide lengthwise freely and guide the scales along each other in a straight line. One of the steel frames is provided at one end with a short stem, terminated by a ball, which is adapted to lie in the recessed jaw of one of the clamps or gauge-holders. The opposite end of the other frame is drilled and tapped to receive the end of a rod of greater or less length, which is also terminated by a ball, and by which the gauge is adapted for specimens of various lengths. The ball at the end of this rod is intended to be fastened in the jaw of the clamp which is at the end of the specimen opposite to that which receives the ball of the short stem.

The lines which make up the scales are exceedingly fine, and are observed through a microscope with a magnifying power of from 50 to 75 diameters. The appearance of the scales, as they are seen in the microscope, is shown in the drawing. The small group of ten lines close together, represents the thousandth's scale, which is marked on one of the pieces of glass, and the longer scale, consisting of lines which seem to be widely separated, is part of the hundredth's scale ruled on the other piece of glass.

If the gauge be attached to a specimen, by fastening the ball-shaped end of one of the gauge stems in its socket in the jaw of the clamp at one end, say the upper end of the specimen, and the ball-like end of the other gauge stem in the jaw of the lower clamp, and if the specimen be stretched, so that the distance between the clamps be increased, then, as the stems and balls are attached to the frames carrying the glasses, these will be made to slide along each other, and the small group of lines of the thousandth's scale will, when observed through the microscope, be seen to travel along the hundredth's scale, and as each succeeding thousandth line passes one of the hundredth's line, movements of one thousandth of an inch will in each case be shown to have occurred; and if any particular line in the thousandth's scale pass from one of the hundredth's line to the next one, then one one-hundredth of an inch movement will be indicated, and these movements correspond to the

elongation of that portion of the specimen which extends between the two planes in which the points of the set-screws lie, by which the two clamps are fastened to the specimen.

The microscope for examining the gauge is fastened to the frame of the testing machine, and a mirror is also fastened to the frame by which light may be thrown through the scales. The eye can easily be educated to estimate tenths of the distances between the lines of the thousandth's scale, with less error than one-half that quantity, so that one ten thousandth of an inch may be read from the gauge with certainty. The steel frames can easily be fitted so accurately that no appreciable change in the relative positions of the two scales will occur from looseness in the fitting. The gauge, in fact, is found to work well in practice, giving satisfactory and consistent results.

One advantage of this gauge over others made for the same purpose is, that when in place it forms, as it were, part of the specimen, and its indications are quite independent of any displacement of the specimen in the machine, and of any distortions of the machine itself. The fact that such displacements and distortions do occur is evident in the use of this gauge, from the necessity which frequently arises of changing the focusing of the microscope in order to keep the lines clearly in view. The arrangements in the Colt's Armory machine are such as to lead us to expect that fewer errors of this kind exist in it than are likely to be found in other less carefully constructed testing machines.

If a good eye-piece micrometer is to be conveniently had, a substitute for the scales can be made by ruling a single line only on each of the pieces of glass. The distances between these lines corresponding to the various strains can then be measured by the micrometer. A slight improvement might be made in the gauge by inclining the lines of the thousandth's scale to those of the hundredth's in such a manner, that by having parallel longitudinal lines ruled on the smaller scale, a "diagonal" scale would be formed from which ten thousandths of an inch could be read directly.

By reading the gauge carefully, the modulus of elasticity of a specimen 10 or 15 inches long can be determined with considerable accuracy. It is the writer's intention to compare the modulus which is obtained from observing the extension of a specimen by tensile strains, with that obtained from noting the deflections of the same specimen under transverse strains. In some cases the elastic limit of the specimen is given in the tables. This value is approximately the least strain which

produced a perceptible permanent set. In observing this the following course was taken :

A strain of 1,000 lbs. was first applied to the specimen, in order to bring everything to their bearings, and the reading of the gauge was taken. Increments of strain were then applied, and the corresponding readings of the gauge observed, until the supposed elastic limit was nearly reached. The strain was then reduced to some weight, which we will call x, at which a gauge reading had previously been noted, and a reading was again taken. If the new reading coincided with the former one, it was assumed that no perceptible permanent set had taken place. A strain somewhat greater than any before used was then applied to the specimen, and after having been maintained for one or two minutes, the strain was once more reduced to x, and the gauge again examined, in order to see whether the reading remained the same as before, and so on until the point was reached at which, when the strain was reduced to x, a reading different to the previous ones was obtained, which showed that a set had been produced.

If a very considerable strain was sustained without set, higher and higher bases of reference were taken to save time ; as for example, after the strain had been increased greatly beyond x, and had been referred back to that base several times, a new base at a higher strain, say y, would be taken, and the effects of strains greater than that would be noted by reducing the strain to y and comparing the reading with former ones at that strain. The process will be better understood by reference to the following table of readings taken in determining the elastic limit of specimen No. 326, "Burden's Best" iron :

Strains.* Lbs.	Gauge.† In.	Strains.* Lbs.	Gauge.† In.
1000.	0.1827.	7000.	0.1860.
6000.	0.1855.	‡–8500.	0.18695.
x.—1000.	0.1827.	7000.	0.18605.
7000.	0.1860.	8750.	0.18705.
y.—6000.	0.1855.	7000.	0.1861.
7500.	0.1862.	9000.	0.1872.
z.—7000.	0.1860.	7000.	0.1862.
8000.	0.1866.	§–9250.	

* On the specimen in the succession in which they were applied.

† Readings corresponding to the different strains.

‡ The elastic limit was here passed, as is shown by the next reading, at 7000 lbs. Here a change of reading of only one twenty thousandth of an inch was noted.

§ With this strain the specimen began to stretch rapidly, and the gauge was removed.

The specimens of bar iron were all turned and polished accurately to size, the most of them from 1½ inch round bars. The finished diameter of the specimens of Lowmoor, Ulster and Salisbury irons was 0.715 of an inch, and the length of the reduced portion of the specimens was 5 inches. All other particulars are given in the tables annexed.

Mr. Collingwood.—In testing iron, careful methods, such as Mr. Richards has described, should be generally introduced. The first thing required is a simple and reliable testing machine. It is certainly a debatable question, whether in most cases it is essential to break the enormous bars of metal, which is sometimes done; in all machines employed for the purpose there are certain sources of error where extreme strains are produced, which tend to impair the results obtained. These, in hydraulic machines, may all be summed up under the head of friction; they do not follow any known law, for the reason that they are enormously increased by distortion of the parts, not only of the machine, but of the mercurial or other gauges which measure the hydraulic pressure. The effect of distortion is more serious than at first sight would seem probable. To test the question, Mr. Colman Sellers made of steel, a very accurately fitted piston gauge. Upon the upper end of the piston was a knife edge which bore against the under side of a weighted lever. The readings were entirely satisfactory for moderate pressures; but from about 2,000 pounds per inch, and upward, they varied in such a manner as to cause him to abandon this method of measurement. Besides the difficulty of measuring the hydraulic pressure, is that of determining how much is lost in overcoming the friction of the ram in the cylinder. This will vary with the condition of the packings, freedom from rust, &c. In a small machine, very nicely made by Tangey Bros., Birmingham, testing to 25 tons, and measuring pressure both by mercurial gauge and weighted levers, the following result was obtained. Everything being in perfect order, 5 tons (gross) strain was applied, or a pressure per square inch on the ram of about 360 pounds. The reading on the lever was one per cent. less at this low pressure than that by the gauge.

In lever machines there is danger of an inaccurate measurement of the relative lengths of lever arms, and this increases as the machine becomes worn by use. Each machine may give results which are consistent and suitable for comparison with each other, but absolute accuracy is required to enable comparisons of results from different machines. In future tests of iron, it seems desirable that attempts should be

made to determine the effect of variations in chemical constitution, and of the repetition of strains alternately tensile and compressive.

Prof. Wood—Most of the experiments on the strength of materials which are presented to us, are to determine the ultimate strength; what we need to know is not this, but the strength within the elastic limit. Fortunately, it has been found that with iron, in almost every case, the elastic limit exceeds one-third of the breaking strength, while the best practice demands that the breaking strength shall be from 4 to 6 times the greatest strain which is to be applied to the piece.

One objection to the ordinary method of testing materials is, that we cannot thereby try the identical pieces used. Selected specimens, or even specimens taken at random, may possess all the desired qualities, while some of the pieces used may be very defective. There ought to be a systematic mode of testing, without injury, the parts which are to make up the structure. I am aware that such has been employed (as in the St. Louis Bridge, in this country, and in large structures in foreign countries), but it is not common. The general practice is, for railroad companies and others to specify that the iron to be put in a bridge, or similar structure, shall have a given tensile strength (often fixed at 60,000 pounds per square inch), without specifying any condition under which the test shall be made.

These experiments of Mr. Richards show that the length of the piece is an important element in the problem, the shorter pieces having a tenacity of about 62,000 pounds, and the longer ones, of but about 49,600 pounds per square inch of original section, while they had nearly the same tenacity per square inch of breaking section. It is evident from this, that contracts which specify the tensile strength of the iron, should also specify the length and diameter of the specimen to be tested. If its length is from 15 to 18 inches, and its diameter one inch, I think it will not be easy to find iron which will then show a tenacity of 60,000 pounds per square inch.

The fact having been established that short specimens appear to be stronger than long ones, it is not difficult to assign a theoretical reason for it. The specimens appear to have drawn out like wax—they behaved therein like viscous substances, as might have been anticipated from M. Tresca's experiments. In the short specimens, the material just outside the neck or smaller portion, and in the immediate vicinity of it, resisted the reduction of the section, and in this way added perceptibility to the strength exhibited.

TENSILE STRENGTH OF SEVERAL KINDS OF BAR IRON.

Tables I and II show the effect of the shape of the specimen on the results.

TABLE I.—EFFECT OF TENSILE STRAIN ON "BURDEN'S BEST" IRON, AS DETERMINED FROM "LONG SPECIMENS."

Test number.	Nominal shape of the specimens.	Original dimensions of the portion stretched.		Least weight which produced a permanent set.	Breaking weight.	Diameter of place of fracture, measured after fracture.	Length of the portion stretched, measured after fracture.	REDUCED RESULTS.					
								TENSILE STRENGTH PER SQUARE INCH		Limit of elastic resistance per sq. in. of original cross section.	Reduction of area of cross section at place of fracture, measured after fracture.	Ultimate elongation referred to original length.	Ratio of elastic resistance to tensile strength.
		Diameter.	Length.					of original cross section.	of fractured cross section.				
		Ins.	Ins.	Lbs.	Lbs.	Ins.	Ins.	Lbs.	Lbs.	Lbs.	Per Ct.	Per Ct.	Per Ct.
324	Long.	0.620	5.0	9500	14860	0.45	6.66	49206	93459	31450	47.	33.	64.
325	Do.	0.619	5.0	8000	14970	0.45	6.55	49734	94218	26580	47.	32.	53.
326	Do.	0.619	5.0	8500	14910	0.45	6.47	49535	93774	28240	47.	29.	57.
327	Do.	0.619	5.0	8000	14810	0.44	6.56	49202	97434	26570	49.	32.	54.
332	Do.	0.711	5.0	10500	19590	0.50	6.69	49345	100000	26460	50.	34.	54.
333	Do.	1.000	5.5	....	39250	0.76	7.00	49974	86454	.....	42.	28.	....
334	Do.	1.000	5.0	....	39210	0.74	6.28	49924	91186	.....	45.	29.	....
335	Do.	1.000	5.0	....	39100	0.73	6.48	49784	93430	.....	47.	26.	....
					Averages for the Long Specimens,			49588	93744	27860	46.7	30.4	56.4

TABLE II.—TENSILE STRENGTH of "BURDEN'S BEST" IRON, AS DETERMINED FROM "SHORT SPECIMENS."

328	Short.	0.614	....	....	18420	0.515	....	62230	90620	....	30.	....	....
329	Do.	0.616	....	....	17960	0.510	....	61670	88040	....	31.	....	....
330	Do.	0.616	....	....	18130	0.520	....	60840	85358	....	29.	....	....
331	Do.	0.615	....	....	17940	0.510	....	60407	87941	....	31.	....	....
336	Do.	1.002	....	....	50340	0.850	....	63840	88783	....	28.	....	....
337	Do.	1.000	....	....	49910	0.850	....	63547	88025	....	28.	....	....
					Averages for the Short Specimens,			62089	88128	....	29.5	....	....

TABLE III.—TENSILE STRENGTH OF "LOWMOOR," "HAMMERED SALISBURY" AND "ULSTER" IRONS, IN CYLINDRICAL SPECIMENS.

The annexed results are averages of 3 "*long*" specimens each of Lowmoor and hammered Salisbury irons, and of 6 "*long*" specimens of Ulster iron.	Lowmoor.........	51062	101362	25400	49.0	28.7	49.7
	Salisbury..........	53420	78605	23340	32.3	20.0	43.7
	Ulster............	49630	100000	23900	50.4	27.0	48.2

TENSILE STRENGTH OF BOILER PLATE.

TABLE IV.—SHOWING THE EFFECT OF THE SHAPE OF THE SPECIMEN ON THE RESULT, FOR "BAY STATE" FLANGE PLATE.

Number of specimens tested.	Kind of Iron and its Brand.	Nominal shape of the specimens.	Direction of the strain relatively to the direction in which the plate was rolled.	Approximate dimensions of the original cross sections.	Tensile Strength per square inch					Reduction of area of cross section at place of fracture, measured after fracture.	Ultimate elongation referred to original length.
					of original cross section.				of fractured cross section.		
					Strongest specimen.	Weakest specimen.	Averages.	General averages.			
				Ins.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Per Ct.	Per Ct.
6	Bay State, Flange	Long.	Lengthwise.	1.25×0.29	47785	46484	47017	47450	64411	27.0	19.3
3	Do.	Do.	Crosswise.	0.75×0.29	49113	46815	47884		56755	14.0	12.1
6	Do.	Short.	Lengthwise.	1.25×0.29	52993	50770	51943	52102	61295	15.3	....
3	Do.	Do.	Crosswise.	0.75×0.29	53161	51597	52262		58170	10.2	....

TABLE V.—BAY STATE PLATE IN "LONG" SPECIMENS.

Number of specimens tested.	Kind of Iron and its Brand.	Nominal shape of the specimens.	Direction of the strain.	Approximate dimensions.	Strongest specimen.	Weakest specimen.	Averages.	General averages.	Of fractured cross section.	Reduction of area.	Ultimate elongation.
14	Bay State, Flange	Long.	Lengthwise.	1.25×0.30	51378	44036	48098	47187	63596	24.8	16.2
12	Do.	Do.	Crosswise.		49023	39898	46277		52349	10.7	10.7
4	Bay State, C. No. 1	Do.	Lengthwise.		48819	46086	47725	46013	55967	14.5	11.5
4	Do.	Do.	Crosswise.		45240	42961	44301		48849	9.2	6.5
4	Bay State, Homogeneous Metal	Do.	Lengthwise.		71139	70100	70672	....	136473	52.0	20.0

TABLE VI.—VARIOUS KINDS OF PLATE IN "SHORT" SPECIMENS.

Number of specimens tested.	Kind of Iron and its Brand.	Nominal shape of the specimens.	Direction of the strain.	Approximate dimensions.	Strongest specimen.	Weakest specimen.	Averages.	General averages.	Of fractured cross section.	Reduction of area.	Ultimate elongation.
2	Thorneycroft, English (from an old boiler),	Short.	Lengthwise.	0.87×0.27	47245	46410	46827	45293	49700	....	....
2	Do.	Do.	Crosswise.		44355	43165	43760		45000	....	....
3	Pennsylvania, "Common,"	Do.	Lengthwise.	0.87×0.16	54699	44581	49227	48434	55604	....	....
3	Do.	Do.	Crosswise.		54031	43436	47641		50000	....	....
3	Pennsylvania, C. No. 1	Do.	Lengthwise.	0.87×0.28	56429	53870	52986	53646	57600	....	....
2	Do.	Do.	Crosswise.		55218	53395	54306		60700	....	....
1	Pennsylvania, Flange	Do.	Lengthwise.		.....	.....	54466	53733	70600	....	....
2	Do.	Do.	Crosswise.		54819	51184	53001		57490	....	....
10	Bay State, C. No. 1	Do.	Lengthwise.		58150	48650	53129	52528	.....	....	....
4	Do.	Do.	Crosswise.		53145	50449	51928		.....	....	....
4	Bay State, Flange	Do.	Lengthwise.		57934	54377	57529	55612	.....	....	....
2	Do.	Do.	Crosswise.		53998	53395	53696		.....	....	....
2	Sligo Fire Box	Do.	Lengthwise.		53791	52546	53168	52750	67100	....	....
2	Do.	Do.	Crosswise.		54394	50272	52333		56400	....	....
1	Do.	Do.	Lengthwise.	1.25×0.33	.....	.....	60911	.....	81700	....	....

NOTE.—In the "long" specimens of boiler plate, the length from which the sketch was measured was, in most cases, 4 inches.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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LXXVI.

ON THE STRENGTH, ELASTICITY, DUCTILITY AND RESILIENCE OF MATERIALS OF MACHINE CONSTRUCTION,

AND ON VARIOUS HITHERTO UNOBSERVED PHENOMENA, NOTICED DURING EXPERIMENTAL RESEARCHES WITH A NEW TESTING MACHINE, FITTED WITH AN AUTOGRAPHIC REGISTRY.

A paper by Prof. R. H. THURSTON, Member of the Society,

READ FEBRUARY 4, 1874.

SECTION I.

1. INTRODUCTORY.*—Some months ago, while engaged with the advanced classes of the Stevens Institute of Technology, in experimental investigations of the resistance of materials, it was found that coëfficients were given, by various authorities, which neither accorded fully with each other or with those then obtained.

The desirability of determining how far these differences were due to errors of observation, and how far to variation in the quality of the materials examined, induced the writer to design several machines for the purpose of conducting with them a more extended and exact series of experiments. The machine for measuring torsional resistance was furnished with an automatic registry, recording a diagram which is a reliable and exact representation of all circumstances attending the distortion and fracture of the specimen. No system of personal observation could probably be devised which could yield results either as reliable or as precise as such a system of autographic registry, and, as no method previously in use had given simultaneously, and at every instant during the test, the intensity of the distorting force and the magnitude of the coincident distortion, it was anticipated that the new method of investigation

* *Vide* Journal Franklin Institute, 1873.

might be fruitful of new and, possibly, important results. This expectation, as will be seen, has been more than realized.

2. Description of the Apparatus.—The machine, as planned by the writer, and as built in the instrument makers' workshop, at the Stevens Institute, is shown in Fig. 1. This form is that with which the investigations to be described were made. Since its construction, in 1872, however, some changes and improvements have been made in the design to adapt it to general work, and new designs have been made for special kinds of work, as for wire mills, railroad shops and bridge building.

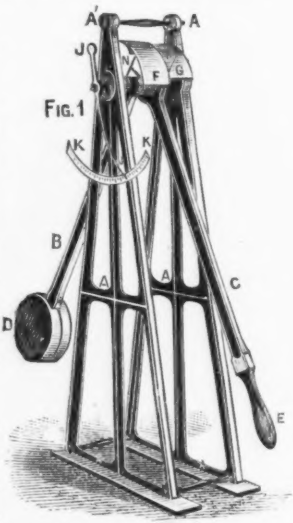

Fig. 1

Two strong wrenches, $C\ E$, $B\ D$, are carried by the frames $A\ A$, $A^1\ A^1$, and depend from axes which are both in the same line, but are not connected with each other. The arm, B, of one of these wrenches carries a weight, D, at its lower end. The other arm, C, is designed to be moved by hand, in the smaller machines, and by a gear and pinion, or a worm gear in larger forms of the apparatus. The heads of the wrenches are made as shown in Fig. 2, the recess, M, being fitted to take the head, on the end of the test pieces, which is usually given the form shown in Fig. 4.

Fig. 2

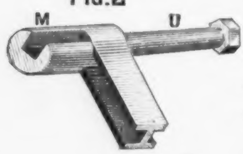

Fig. 3

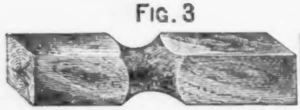

Fig. 4

A guide curve, F, of such form that its ordinates are precisely proportional to the torsional moments exerted by the weighted arm, $B\ D$, while moving up an arc to which the corresponding abscissas of the curve are proportional, is secured to the frame $A\ A^1$. The pencil holder, J, is carried on this arm, $B\ D$, and as the latter is forced out of the vertical position, the pencil is pushed forward by the guide curve, its movement

being thus made proportionate to the force which, transmitted through the test piece, produces deflection of the weighted arm. This guide line is a curve of sines. The other arm, *C E*, carries the cylinder, *G*, upon which the paper receiving the record is clamped, and the pencil, *J*, makes its mark on the table thus provided. This table having a motion, relatively to the pencil, which is precisely the angular relative motion of the two extremities of the tested specimen, the curve described upon the paper is always of such form that the ordinate of any point measures the amount of the distorting force at a certain instant, while its abscissa measures the distortion produced at the same instant. The maximum hand, *J*, is sometimes useful as a check upon the record of maximum resistance.

The convenience of operation, the small cost,* and the portability of the machine are hardly less important to the engineer than the accuracy, and the extraordinary extent of information obtainable by it.

3. Method of Operation.—The test piece having been given the shape and size which are found best suited for the purposes of the experiment, and to the capacity of the machine, it is placed in the jaws of the two wrenches, each of which takes one of its squared ends, and, a force being applied to the handle, *E*, the strain thrown upon the specimen is transmitted through it to the weighted arm, *B D*, causing it to swing about its axis until the weight exerts a moment of resistance which equilibrates the applied force. As the magnitude of the distorting force changes, the position of the weight simultaneously changes, and the pencil indicates, at each instant, the value of the stress upon the test-piece. As the piece yields under strains of increasing amount, also, the pencil is carried in the direction of the circumference of the cylinder on which its record is made, and to a distance which is proportional to the amount of distortion, *i.e.*, to the "total angle of torsion." As the applied force increases, the specimen yields, and finally, rupture occurring, the pencil returns to the base line, at a distance from the starting point which measures the angle through which the test piece yielded before its fracture became complete.

4. Interpretation of the Diagrams.—It has been shown that the vertical scale of the diagrams produced is a scale of torsional moments, and that the horizontal scale is one of total angles of torsion. Since the resistance to shearing, in a homogeneous material, varies with the resist-

* Machines of the size of that used in these experiments, but of improved design, are made at the Stevens Institute, at prices as low as $150.

ance to longitudinal stress, it follows that the vertical scale is also, for such materials, a scale of direct resistance, and that, with approximately homogeneous substances, this scale is approximately accurate, where, as here, all specimens compared are of the same dimensions. Since the elasticity of the material is measured by the ratio of the distorting force, to the degree of temporary distortion produced, the diagrams obtained will exhibit the elastic properties of the material, as well as measure its ductility and its resilience.

Referring to the diagrams shown in the accompanying plates, it will be noticed that the first portion of the line is a curve of small radius, convex toward the axis of abscissas, and that the line then rises at a slight inclination from the vertical, but becoming very nearly straight, until, at a point some distance above the origin, it takes a reversed curvature. The first portion of the line is probably formed by the yielding of the loosely fitted packing pieces securing the heads of the specimen, and, after they have taken a bearing, by the early yielding, in some materials, of particles already overstrained. When a firm hold is obtained, the line becomes sometimes nearly straight, and the amount of distortion is seen to be approximately proportional to the distorting force, illustrating "Hooke's law," *Ut tensis sic vis.*

After a degree of distortion which is determined by the specific character of each piece, the line becomes curved, the change of form having a rate of increase which varies more rapidly than the applied force. When this change commences, it seems probable that the molecules, which, up to that point, retain generally, their original distribution, while varying their relative distances, begin to change their positions with respect to each other, moving upon each other in a manner similar, probably, to that action described by Mon. Tresca, and called the "Flow of Solids," * and to which attention has already been called by Prof. J. Thompson.†

It is this point, at which the line commences to become concave toward the base, that *is considered to mark the "limit of elasticity."* It will be noticed that it is well defined in experiments upon woods, is less marked, but still well defined in the "fibrous" irons and the less homogeneous specimens of other metals, and becomes quite indeterminable with the most homogeneous materials, as with the best qualities of well worked cast-steel. This point does not indicate the first "set,"

* L'Ecoulement des Corps Solides; Paris, 1869, 1871.

† Cambridge and Dublin Mathematical Journal, Vol. III, 1848, pp. 252-266.

since, as will be hereafter seen, a set is found to occur, either temporary or permanent, and usually partly temporary and partly permanent, with every degree of distortion, however small. It is at this "elastic limit" that the sets begin to become considerable in amount and almost wholly permanent.

The inclination of the straight portion of the line from the vertical measures the *stiffness* of the specimen, the quantity $Cot.\ \Theta = \frac{1}{Tan.\ \Theta}$ being the ratio of the distorting force to the amount of distortion up to the "limit of elasticity." As it would seem from the results of experiment, as well as of deduction, that this rigidity is very closely, if not precisely, proportional to the hardness, in homogeneous substances, this quantity $Cot.\ \Theta$ may be taken, for practical purposes, as a measure of the hardness of the metals, as well as of their elastic resistance to compression.

After passing the elastic limit, the line becomes more and more nearly parallel to the base line, and then, with the woods invariably, and in some cases with the metals, begins to fall rapidly before fracture becomes evident in the specimen. Where the rising portion of the line turns and becomes nearly parallel with the axis of abscissas, the viscosity of the material is such that the outer particles "flow" upon those within, and, while themselves still offering maximum resistance, permit molecules nearer the axis to also resist with approximately maximum force. It seems probable that, with the more ductile substances, nearly all are brought up to a maximum in resistance before fracture occurs, and this circumstance will be seen hereafter to have an important influence in determining the resistance to rupture. The hardest and most brittle materials break, with a snap, before any such flow becomes perceivable, and before the line of the diagram commences to deviate, in the slightest degree, from the direction taken at the beginning, and before the approach to the elastic limit is indicated. It is evident that the standard formulas for torsional, as well as for other forms of resistance, cannot be perfectly correct, since they do not exhibit this difference in the character of the resistance offered by ductile and by rigid materials.

The *elasticity* of the material is determined by relaxing the distorting force, at intervals, and allowing the specimen to relieve itself from distortion so far as its elasticity will permit. In such cases, the pencil will be found to have traced a line resembling, in its general form and position, in respect to the coördinates, that forming the initial portion of the diagram, but almost absolutely straight, and more nearly vertical.

The degree of inclination of this line indicated the elasticity, precisely as the initial straight line was made to give a measure of the original stiffness of the test piece, the cotangent of the angle made with the vertical. $Cot.\ \Phi = \frac{1}{Tan.\ \Phi}$ being the ratio of the force required to spring the piece through the range recoverable by elasticity, to the magnitude of that range. The fact, to be shown, that this value is always greater than $Cot.\ \Theta$, for the same metal is evidence that more or less permanent set will always occur, and that the original stiffness of the specimen is always modified, whatever the magnitude of the applied force. The form of the line of elastic change indicates also the character of the molecular action producing it.

Finally, the form of the curve after passing the maximum, or after passing the point at which fracture commences, exhibits the method of variation of strength during the process of fracture. This portion is very difficult to obtain, with even approximate accuracy, with any but the toughest and most ductile materials. This terminal portion of the diagram would be, theoretically, a cubic parabola, the loss of resisting power varying with the progressive rupture of concentric layers, and the remaining unbroken cylindrical portion becoming smaller and smaller until resistance vanishes with the fracture of the axial line. In some cases, the curves obtained from ductile metals exhibit this parabolic line very distinctly. In all hard materials, the jar produced by the sudden rupture of surface particles is sufficient to separate those within, and the terminal line is straight and vertical.

The *homogeneity* of the material tested is frequently hardly less important than its strength, and it is very desirable to obtain evidence which may enable the experimenter to determine the value of tests of samples as indicative of the character of the lot from which the specimens may have been taken. If the specimens are found to be perfectly homogeneous, it may be assumed with confidence that they represent accurately the whole lot. If the samples are irregular in structure and in strength, no reliable judgment of the value of the lot can be based upon their character, and there can be no assurance that, among the pieces accepted, there may not be untrustworthy material which may possibly be placed just where it is most important to have the best. It is evident that the more homegeneous a material, the more regularly would changes in its resistance take place, and the smoother and more symmetrical would be the diagram. The depression of the line immediately after

passing the elastic limit exhibits the greater or less homogeneousness of the material. The fact is illustrated in a striking manner in some of the curves presented, and we thus have—what had never, I believe, been before found—this method of determining homogeneousness.

The *resilience* of the specimen is measured by the area included within its curve, this being the product of the mean force exerted into the distance through which it acts in producing rupture, *i.e.*, it is proportional to the work done by the test piece in resisting fracture, and represents the value of the material for resisting shock. The area taken within the ordinate of the limit of elasticity, measures the capacity for resisting shock without serious distortion or injurious set.

The *ductility* of the specimen is deduced from the value of the total angle of torsion, and the measure is the elongation of a line of surface particles, originally parallel to the axis, which line assumes a helical form as the test piece yields, and finally parts at or near the point where the maximum resistance is formed. Its value is given on Plates II and III for each ten degrees of arc. Since, in this case, there is no appreciable reduction of section, or change of form, in the specimen, this value of elongation is our actual measure of the maximum ductility of the material, and is an even more accurate indication than the area of fractured cross section as usually measured after rupture by tension. It is to be understood that wherever comparisons are here made, without the express statement of other conditions, that specimens of the same dimensions are always represented in the diagrams.

5. Description of Illustrated Diagrams. The Woods.—Plates I and II exhibit sets of curves which illustrate the general characteristics of a large number of materials, the first showing the peculiarities noted during experiments on the woods, and the second giving an interesting comparison of the metals.

The woods experimented upon were the following, the numbers of the respective curves on Plate I, indicating the material here correspondingly marked :—

1. White pine (*Pinus Strobus*).
2. Southern pine (*Pinus Australis*), sap wood.
3. Southern pine, heartwood.
4. Black spruce (*Abies Nigra*).
5. Ash (*Fraxinus Americanus*).
6. Black walnut (*Juglans Nigra*).
7. Red cedar (*Juniperus Virginianus*).

8. Spanish mahogany *(Swietenia Mahogani)*.
9. White oak *(Quercus Alba)*.
10. Hickory *(Carya Alba)*.
11. Locust *(Robinia Pseudo-acacia)*.
12. Chestnut *(Castanea Vesca)*.

The specimens were all of the form shown in Fig. 3, three and three-fourths inches long, with a diameter of neck of seven-eighths of an inch.

It will be noticed that, in all cases, at the commencement of the line, it rises, at a slight inclination from the vertical, and almost perfectly straight. This confirmation of Hooke's law, within the limit of elasticity, is best shown in the detached portion *a*, *a*, *a*, of the curve obtained with locust, in which the horizontal scale is somewhat magnified. The distortion is seen to be very precisely proportional to the distorting force, until the law changes at the limit of elasticity.

It will be observed that, in the larger number of cases, the torsional resistance increases with great regularity nearly to the angle of maximum stress where, suddenly, this rapid rate of increase ceases, and the limit of elastic resistance being passed, resistance diminishes rapidly with further increase of angular movement, until it becomes zero. In the tougher and more dense varieties, this decrease of resistance occurs less slowly, and in some cases only disappears after a large angle of torsion is recorded. In the curves of exceptionally strong and tough woods, in which there is known to exist a great excess of longitudinal over lateral cohesion, as in those of black walnut 6, 6, locust 11, 11, and especially in those of hickory 10, 10, a peculiarity is perceivable which is somewhat remarkable, and which is especially important in a connection to be hereafter referred to at length.

In these instances the resistance is proportional to the amount of torsion, until a maximum is reached, the line then falls as torsion continues, until a minimum is passed, the curve then again rising and passing another maximum before finally commencing an unintermitted descent to the axis of abscissas. Where the difference between longitudinal and lateral cohesion is exceptionally great, the second maximum may, as illustrated, for example, by the line described in recording the test of hickory, have a higher value even than the first. This interesting and previously unanticipated peculiarity was shown, by careful observation, to be due to the sudden yielding of lateral cohesion when the torsional moment reached the value indicated by the first minimum.

The fibres being thus loosened from each other, this loose bundle of filaments yielded readily, until, by lateral crowding as they assumed a helical form and enwrapped each other, their slipping upon each other was gradually checked, and resistance again commenced increasing.

At the second maximum, yielding again began in consequence of the breaking of fibres under the longitudinal stress measured by that component of torsional force having a direction parallel with the filaments in their new positions, the exterior surface threads parting first under this tensile stress, and rupture progressing by the yielding of layer after layer, until the axial line being reached, resistance vanished. In this case, rupture seems never to occur by true shearing along one defined transverse plane. This feature of depression in the curve, occurring as described, is therefore the indication of a lack of symmetry in the distribution of resisting forces. It is evident that it may occur either by a difference in the value of cohesive force in the lateral and longitudinal directions, or by the structural defects of a specimen in which the substance itself may be endowed with cohesion of equal intensity in all directions.

The curves shown in Plate I exhibit well the relative values of these materials for the various purposes of the engineer.

White pine, 1, 1, 1, is shown by the considerable inclination of the line of stiffness from the vertical, to be soft and deficient in rigidity. The limit of elasticity is quickly reached, and the maximum resistance of the specimen is found at 15½ foot-pounds of moment. Rapidly losing strength after passing the limit of resistance, it is entirely broken off at an angle of 130°. The small area comprised by the diagram proves its deficiency of resistance, and its inability to sustain shock.

Yellow pine, 2, 2, 2, 3, 3, 3, far excels the first in all valuable properties shown by the curve. The sapwood seems, in the specimens tested, equally stiff with the heart, but it reaches the elastic limit sooner. The general form of the diagram is the same in both, and is characteristically different from that of the white pine. It evidently has great value wherever rigidity, strength, toughness and resilience are desired in combination with lightness, the latter most important quality, together with their cheapness, aiding the qualities here shown in determining the application of these woods so extensively for general purposes. It should be noted that, since all comparisons of strength are based on measures of volume, a comparison of densities should usually be obtained to assist the judgment in making a choice from among materials of which tests have been made.

Spruce, 4, 4, 4, while possessing far less stiffness than even white pine, excels it somewhat in strength, passing its maximum at 18 foot-pounds, and submitting to a torsion of nearly 200°. It is proven to possess, proportionally greater resilience also. It is, however, far inferior to the yellow pine in every respect.

Ash, 5, 5, 5, is more deficient in strength and toughness than is generally supposed, and rapidly loses its power of resistance after passing the maximum, which point is found at about 27½ foot-pounds. These specimens may have been of exceptionally poor quality, or, possibly, were over-seasoned.

Black walnut, 6, 6, 6, is remarkably stiff, strong and resilient, its diagram resembling somewhat that of oak in general form and dimensions. The maximum of resistance reaches 35 foot-pounds, and the most ductile specimen was only broken off after yielding through an arc of 220°. Its stiffness is shown by the fact that it required a moment of 25 foot-pounds to spring it 10°, yellow pine requiring but 22 foot-pounds and spruce but 8, to give them the same amount of distortion.

Red cedar, 7, 7, 7, is very stiff, but is brittle and deficient in strength, breaking off at 92°, and having a maximum power of resistance of but 20½ foot-pounds. It is, however, one of the stiffest of the woods, its specimen requiring 20 foot-pounds of torsional moment to produce a total angle of torsion of but 5°.

Spanish mahogany, 8, 8, 8, is both strong and stiff, bearing a stress of 44 foot-pounds, and requiring 32 to produce torsion of 10°.

White oak, 9, 9, 9, exhibits less strength than either good mahogany, locust or hickory, but it is exceedingly tough and resilient. Passing the maximum at an angle of 15°, under a torsional stress of 35½ foot-pounds, it retains its power of resistance nearly unimpaired up to about 70°, and then slowly yields until it suddenly gives way, after passing the angle 250°, under a strain due to 9 foot-pounds, and breaks off completely at 253°. This strength, toughness and endurance, under strains due to impact, may be attributed to its considerable lateral cohesion, and to the interlacing of its tenacious fibres, which gives this wood its "cross" grain.

Hickory, 10, 10, 10, has the highest maximum found during these experiments, the second of the pair of maxima already referred to being considerably above the maximum of locust even. This specimen exhibits well the well-known valuable properties of the material, requiring 45 foot-pounds to twist it 10°, reaching a limit of elasticity at 54 foot-

pounds and 13 , and having a maximum resisting moment of 59½ foot-pounds. When it finally yields, it does so quite rapidly, breaking off at 145 .

Locust, 11, 11, 11, gives an excellent diagram. It is the stiffest of all, yielding but 10 at its maximum of 55 foot-pounds, and one piece, which was unusually hard and compact, requiring 48 foot-pounds to distort it 4 , and reaching a maximum angle of torsion of nearly 190 .

It was noticed, during this series of experiments, that different specimens of the same species of wood usually exhibited very nearly equal strength and rigidity, and that marked differences were only occasionally noted in elasticity and resilience.

6. THE METALS, AND THE CURVES PRODUCED BY THEM.—Plate II exhibits a series of curves which illustrate well the general characteristics and the peculiarities of representative specimens of the principal varieties of useful metals. In some cases two specimens have been chosen for illustration, of which one presents the average quality, while the other is the best and most characteristic of its class.

The diagrams obtained by testing metals are quite different in general character from those registered in experiments on the woods, yet there are some points of resemblance which it will be instructive to notice, since these similar characteristics indicate similar properties of the two materials, and a comparison aids greatly in the interpretation of the diagrams. The woods have a structure which differs, in a distinguishing degree, both in the distribution of the substance and in the action of those molecular forces capable of resisting rupture, from that of the metals, the latter being far more homogeneous, in both respects, than the former. Wood consists of an aggregation of strong fibres, lying parallel, or approximately so, and held together often by a comparatively feeble force of lateral cohesion. The latter force being, as often happens, destroyed, the mass becomes a collection of loose threads having the general character of a rope or cord, with slight or no twist. The metals, on the other hand, are naturally homogeneous, both in structure and in the distribution and intensity of the molecular forces. Well-worked and thoroughly annealed cast-steel, as an example, is equally strong in all directions, is perfectly uniform in its structural character, and is almost absolutely homogeneous as to strain. It would be expected, therefore, that the diagrams obtained by breaking such a material would differ from those of the woods, in having a smoother and more regular form, and this is shown to be actually the case by observation of the curves of

cast-steel, cast-iron, bronze and others of the more homogeneous metals and alloys.

Some of the metals, it will be noticed, yield diagrams of less regular form. Wrought-iron, as usually made, has a somewhat fibrous structure, which is produced by particles of cinder, originally left in the mass by the imperfect work of the puddler while forming the ball of sponge in his furnace, and which, not having been removed by the squeezers or by hammering the puddle ball, are, by the subsequent process of rolling, drawn out into long lines of non-cohering matter, and produce an effect upon the mass of metal which makes its behavior, under stress, somewhat similar to that of the stronger and more thready kinds of wood. In the low steels, also, in which, in consequence of the deficiency of manganese accompanying, almost of necessity, their low proportion of carbon, this fibrous structure is produced by cells and "bubble holes" in the ingot, refusing to weld up in working, and drawing out into long microscopic, or less than microscopic, capillary openings.

In consequence of this structure we find, as we should have anticipated, a depression interrupting the regularity of their curves, immediately after passing the limit of elasticity, precisely as the same indication of the *lack of homogeneousness of structure* was seen in the diagrams produced by locust and hickory.

The presence of internal strain constitutes an essential peculiarity of the metals which distinguishes them from organic materials. The latter are built up by the action of molecular forces, and their particles assume naturally, and probably invariably, positions of equilibrium as to strain. The same is true of naturally formed organic substances. The metals, however, are given form by external and artificially produced forces. Their molecules are compelled to assume certain relative positions, and those positions may be those of equilibrium, or they may be such as to strain the cohesive forces to the very limit of their reach. It even frequently happens, in large masses, that these internal strains actually result in rupture of portions of the material at various points, while in other places the particles are either strongly compressed, or are on the verge of complete separation by tension. This peculiar condition must evidently be of serious importance, where the metal is brittle, as is illustrated by the behavior of cast-iron, and particularly in ordnance. Even in ductile metals it must evidently produce a reduction in the power of the material to resist external forces. This condition of internal strain may be relieved by annealing hammered and rolled metals, and by cooling cast-

ings very slowly, in order that the particles may assume, naturally, positions of equilibrium. In tough and ductile metals, internal strain may be removed by heating to a high temperature and then cooling under the action of a force approximately equal to the elastic resistance of the substance. This process, called "Thermo-tension," was first used by Professor Johnson in the course of his experiments as a member of a Committee of the Franklin Institute, in 1836,* and the effect of this action in apparently strengthening the bars so treated, was stated in the report of the committee. The fact that this effect was very different with different kinds of iron was also noted, but it does not appear that the cause of this, which they term "an anomalous" condition of the metal was discovered by them.

Metals which are very ductile may frequently be relieved of internal strain, also, by simply straining them while cold to the elastic limit, and thus dragging all their particles into extreme positions of tension, from which, when released from strain, they may all spring back into their natural and unstrained positions of equilibrium. This fact, which does not seem to have been previously discovered by investigators of this subject, will be seen to have an important bearing upon the resisting power of materials, and upon the character of all formulas in which it may be attempted to embody accurately the law of resistance of such materials to distorting or breaking strain.

Since straining the piece to the limit of elasticity brings all particles subject to this internal strain into a similar condition, as to strain, with adjacent particles, it is evident that indications of the existence of internal strain, and through such indications a knowledge of the value of the specimen, as affected by this condition, must be sought in the diagram, before the sharp change of direction which usually marks the position of the limit of elasticity is reached. As already seen, the initial portion of the diagram, when the material is free from internal strain, is a straight line up to the limit of elasticity. A careful observation of the tests of materials of various qualities, while under test, has shown that, as would, from considerations to be stated more fully hereafter, in treating of the theory of rupture, be expected, this line, *with strained materials, becomes convex towards the base line*, and the form of the curve, as will be shown, is parabolic. The initial portion of the diagram, therefore, determines readily whether the material tested has been subjected

* Journal Franklin Institute, 1836–7.

to internal strain, or whether it is homogeneous as to strain. This is exhibited by the *direction* of this part of the line as well as by its form. The existence of internal strain causes a loss of stiffness, which is shown by the deviation of this part of the line from the vertical to a degree which becomes observable by comparing its inclination with that of the line of elastic resistance, obtained by relaxing the distorting force—*i.e.*, the difference in inclination of the initial line of the diagram and the lines of elastic resistance, *e, e, e*, indicates the amount of existing internal strains.

7. Forged Iron.—In Plate II, the curves numbered 6, 1, 22 and 100, are the diagrams produced by three characteristic grades of wrought-iron. The first is a quality of English iron, well known in our market as a superior metal. The second is one of the finest known brands of American iron, and the third is also of American make, but it does not usually come into the market in competition with well known irons, in consequence of the high price which is consequent upon the necessary employment of an unusual amount of labor, in securing its extraordinarily high character.

No. 6 at first yields rapidly under moderate force, only about 50 foot-pounds of torsional moment being required to twist it 5°. It then rapidly becomes more rigid, as the internal strains, so plainly indicated, are lost in this change of form, and at 6° of torsion, the resistance becomes 60 foot-pounds, as measured at *a*. Here the elastic limit is reached. The next 3° produce no increase of resistance. This fact shows that this iron, which was not homogeneous as to strain, is also not homogeneous in structure. We conclude that it must be badly worked and seamy, and that it may have been rolled too cold; the former is the probable reason of its lack of homogeneous structure, the latter gave it its condition of internal strain. After the first 9° of torsion, resistance steadily rises to a maximum, which is reached only when just on the point of rupture, and the piece finally commences breaking at 250°, and is entirely broken off at 285°. Its maximum elongation, whose value is proportionable to the reduction of section noted with the standard testing machines, is 0.691. The terminal portion of the line, after rupture commences, is not usually accurate as a measure of the relation of the force to the distortion. The increase of resistance between the angle 9° and the angle of rupture is produced by the additional effort in resistance due to the "flow" or drawing out of particles, as already indicated, and the precise effect of

which will be noticed at length in a succeeding section relating to the theory of rupture.

Applying the scale for tension, which in the case of these curves was very exactly 24,000 pounds per square inch for each inch measured vertically on the diagram, we find that the elastic limit was passed under a stress equivalent to a tension of 19,800 pounds per square inch, and that the ultimate tenacity was 59,200 pounds per square inch. When nearly at the maximum the specimen was relieved from stress, the pencil descending to the base line, and the elasticity of the piece produced a certain amount of recoil. The angle intercepted between the foot of this nearly vertical line, *e*, and the origin at o, measures the *set*, which is almost entirely permanent. The distance measured from the foot of the perpendicular, let fall upon the axis of abscissas, from the head of this line to the foot of the line *e*, measures the elasticity, *and is inversely proportional to the modulus.* A comparison of the inclination of the line made by the pencil in reascending, on the renewal of the strain with the initial line of the diagram, gives the indication of the amount of internal strain originally existing in the piece.

It will be noticed that the horizontal movement of the pencil is recommenced at *l*, under a higher resistance than was recorded before the elastic line was formed. In this case the piece had been left under strain for some time before the stress was relieved, and the peculiarity noted is an example of an increase of resistance under stress,* or more properly of *the elevation of the elastic limit*, of which more marked examples will be shown subsequently.

The exceptional stiffness and limited elastic range here shown, as compared with the other examples given, is probably a phenomenon accompanying and due to this increase of resistance under stress.

Examining No. 1 in a similar manner, we find that it is far freer from internal strain than No. 6, its initial line being much more nearly straight and rising more rapidly. It is rather less homogeneous in structure, and is forced through an arc of 6°, after having passed its elastic limit, before it begins to offer an increasing resistance. It is evidently a better iron, but less well worked, and, as shown by the position of the elastic limit, is somewhat harder and stiffer. No. 1 retains its higher resistance quite up to the point at which No. 6 received its incidental accession of resistance by standing under strain, and the two pieces break at, practically, the same point, No. 1 having slightly the

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* *Vide* Transactions, Vol. II, page 290.

greater ductility. When the "elastic line," e, is formed, just before fracture, it is seen that No. 1 has a greater elastic range and a lower modulus than No. 5. It should be observed that the line by which the pencil *descends* to the base line has usually no value, owing to the fact that no care is generally taken to remove the stress as gradually as it is applied. When such care is taken, the lines are usually coincident, and do not form the loop here seen. It will also be noticed that these lines often cross each other, that on the right being the important line. The elastic line formed by No. 1 at between 40° and 45° of torsion is seen to be very nearly parallel with that obtained near the terminal portion of the diagram, and illustrates the fact here first revealed to the eye, that *the elasticity of the specimen remains practically unchanged up to the point of incipient rupture*, and this fact corroborates the deductions of Wertheim* and others who came to this conclusion from less satisfactory modes of research. All experiments yet made give a similar result.

No. 22 illustrates the characteristics of a metal which probably represents one of the best qualities of wrought iron made in this or in any other country, and with which every precaution has been taken to secure the greatest possible perfection, both in the raw material and in its manufacture. The fact that it finds a market at sixteen cents a pound proves that even such care and expense are well applied. The line of this diagram, starting from O, rising with hardly perceptible variation from its general direction, turns, at the elastic limit, a, under a moment of about 80 foot-pounds, equivalent to a tension of about 24,000 pounds per square inch; and with between 2° and 3° of torsion only, and thence continues rising in a curve almost as smooth and regular as if it had been constructed by a skilful draughtsman. Reaching a maximum of resistance to torsion of 220 foot-pounds and an equivalent tensile resistance of over 66,000 pounds per square inch, at an angle of 345°, it retains this high resistance up to the point of rupture some 358° from its starting point. The maximum elongation of its exterior fibres is 1.2, making them at rupture 2.2 times their original length. This would produce a probable breaking section in the common testing machine equal to 0.4545 of the original section.†

From the beginning to the end this specimen exhibits its superiority, in all respects, over the less carefully made irons, Nos. 1 and 6, which, it should be remembered, are themselves deservedly known as good brands.

* *Vide* Annals de Chimie et de Physique.

† Compare Kirkaldy; Strength of Iron and Steel; pp. 111, 135, for reduction in Yorkshire and Swedish bars. The elongation there given has, of course, no value as a measure of ductility.

The homogeneousness of No. 22 is almost perfect, both in regard to strain and to structure, the former being indicated by the straightness of the first part of the diagram and its parallelism with the "elastic line," *e*, produced at 217½°, and the latter being proven by the beautiful accuracy with which the curve follows the parabolic path indicated by our theory as that which should be produced by a ductile homogeneous material. At similar angles of torsion, No. 22 offers invariably much higher resistance than either Nos. 1 or 6, and this superiority, uniting with its much greater ductility, indicates an immensely greater resilience. It is evident that for many cases, where lightness combined with capacity to carry live loads and to resist heavy shocks are the essential requisites, this iron would be by far preferable, notwithstanding the cost of its manufacture, to any of the cheaper grades. Comparing their elasticities, as shown at 210°, 215°, it is seen that No. 22 is about equally stiff and elastic with No. 1, while both have a wider elastic range and are less rigid, and hence are softer than No. 6, whose elastic line is seen at 221°. All of the characteristics here noted can be accurately gauged by measuring the diagrams, and constants are readily obtained for all formulas, as illustrated in a later section of this paper, in which the construction of formulas and the determination of constants will be made the subject of investigation.

No. 100 is the curve obtained from a piece of Swedish iron, marked GF. Its characteristics are so well marked that one familiar with the metal would hardly fail to select this curve from among those of other irons. Its softness and its homogeneous structure are its peculiarities. Its curve, at first, coincides perfectly with that of No. 6. It has, however, slightly less of the condition of internal strain, and a somewhat higher limit of elasticity. The elastic limit is found at 5½° of torsion, and at a stress of 65 foot-pounds of moment, equivalent to 19,500 pounds on the square inch, in tension. Its increase of resistance, as successive layers are brought to their maximum and begin to flow, is very nearly the same as that of the specimens Nos. 1 and 6, and the line lies between the diagrams given by these irons up to 30°, and then falls slightly below the latter. At 220°, it attains a maximum resisting power, and here the outer surface begins to rupture, after an ultimate stretch, of lines formerly parallel to the axis, amounting to 0.564. Had this elongation taken place in the direction of strain, as in the usual form of testing machine, it would have produced a reduction of section to 0.64, the original area.* At this

*Compare Styffe; Strength of Iron and Steel; p. 133, Nos. 26–30.

point the stress in tension equivalent to the 176 foot-pounds of torsional stress, is 52,800 pounds per square inch. From 250° the loss of resistance takes place rapidly, but the actual breaking off of the specimen did not occur until it had been given a complete revolution. This part of the diagram distinguishes the metal from all others, and shows distinctly the exceptionally tough, ductile and homogeneous character which gives the Swedish irons their superiority in steel making. No. 22, even, although much more more extensible, is harder than No. 100, and yields more suddenly when it finally gives way.

A comparison of the results here recorded with those obtained by Styffe,* will afford a good basis upon which to form an idea of the accuracy as well as the convenience of this method of deriving them. An examination of the broken test piece gives some evidence confirmatory of the record. The exterior surface of the twisted portion has an appearance intermediate between that of No. 1, Fig. 5,† and No. 22, Fig. 7, with an evident tendency to "kink." The surface of fracture is lighter and more lead-like than even No. 22, and its "fibre" is finer and texture more plastic in appearance. It is beautifully uniform in character. On one end of this specimen, where a piece had been nicked and then broken off by a sharp blow, the absence of all fibrous appearance, and the granular texture and magnificently fine, regular grain are very marked, and indicate that the material is entitled to its established position as the purest metal known in the market. The specimens themselves furnish almost as valuable information, after test, as the diagrams contain, and should always be carefully inspected with a view to securing additional or corroborative information. Fig. 5 is a sketch of specimen No. 1, and shows its somewhat granular fracture, and the seamy structure produced by a defective method of working. Fig. 6, from specimen No. 16, more nearly resembles

Fig. 5.

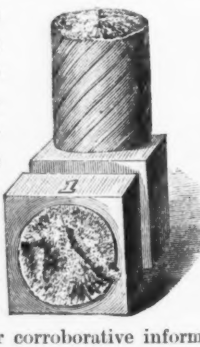
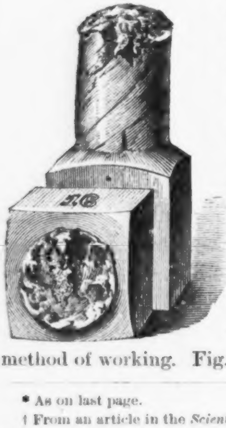


Fig. 6.



* As on last page.

† From an article in the *Scientific American*, of January 17th, 1874, on "Testing the Quality of Iron, Steel and other Metals without Special Apparatus."

that which gave the diagram marked 6. The metal is seen to be good, tough, and better in quality than No. 1, but it is even more seamy, and even less thoroughly worked, as is evidenced by the cracks extending around the neck, and by the irregularly distributed flaws seen on its end.

Fig. 7.

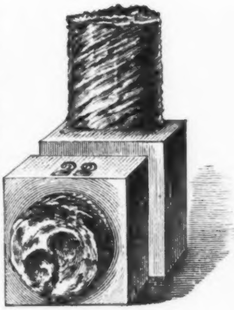


Fig. 7 exhibits the appearance of No. 22 after fracture, and shows, even more perfectly than the penciled record, the splendid character of the material. The surface of the neck was originally smoothly turned and polished, and carefully fitted to gauge. Under test it has become curiously altered, and has assumed a rough, striated appearance, while the helical markings extend completely around it. The end has the peculiar appearance which will be seen to be characteristic of tough and ductile metals, and the uniformly bright appearance of every particle in the fractured section shows how all held together up to the instant of rupture, and that fracture finally took place by true shearing. Rupture by torsion thus brings to light every defect and reveals every excellence in the specimen. Rupture by tension rarely reveals more than the mere strength of the material.

8.—Low Steels.—In Plate II, and above the curves just described, are a set obtained during experiments on "low steels," produced by the Bessemer and Siemens-Martin processes. In general character, the curves are seen to resemble those of the standard irons, as illustrated by Nos. 1 and 6. The irons contain usually barely a trace of carbon. These steels contain from one-half to five-eighths of one per cent. The irons are made by a process which leaves them more or less injured by the presence of impurities, from which the utmost care can seldom free them. The steels are made from metal which has been molten and cast, a process which allows a far more complete separation of slag and oxides. The low steels, however, are liable to an objectionable amount of porosity, due to the liberation of gas while the molten mass is solidifying, whenever the spiegeleisen, employed as a conveyor of carbon, is not very rich in manganese. The results of these differences in constitution and treatment are readily seen by inspecting the curves. They show a stiffness equal to No. 6, and about the same degree of internal strain. They contain a sufficient number of the capillary channels,

produced by drawing down the pores while working the ingot into bar, to cause a lack of homogeneousness in structure, very similar to that produced in iron by cinder. They have a much higher elastic limit, and greater strength, and the softer grades have great ductility. In resilience, these softest steels excel all other metals, except the unusual example, No. 22, and are evidently the best materials that are now obtainable for all uses where a tough, strong, ductile metal is needed to sustain safely heavy shocks. A comparison of the diagrams of two competing metals may thus be made to indicate how far a difference in price should act as a bar to the use of the costlier one. For many purposes, a metal having double the resilience of another is worth more than double-price. For general purposes, a comparison of the resilience of the metals within the elastic limit is of supreme importance. No. 6 is seen to have more resilience within this limit than No. 1, and the steels far more than either; but No. 1 would take a set of considerable amount far within the true elastic limit, as indicated at *a*. The most valuable measure is obtained by determining the area intercepted between the "elastic line" and the perpendicular let fall from its upper end; this measures the resilience of elastic resistance, which is the really important quality.

No. 98 was cut from the head of an English Bessemer rail made from unmixed Cumberland ores. It contains nearly 0.4 per cent. carbon. It is quite homogeneous, has a limit of elasticity at 88 foot-pounds of torsional, or 26,400 pounds per square inch tensile stress, approaches its maximum of resistance rapidly, and, at 210°, the torsional moment becomes 225 foot-pounds, equivalent to 67,500 pounds per square inch tensile stress. It only breaks after a torsion of 283°, and with an ultimate elongation of 80 per cent, equivalent to a reduction of cross section to 0.556.

No. 76 is a Siemens-Martin steel made from mixed Lake Superior and Iron Mountain ores, and contained about the same amount of carbon as the preceding. It contains rather more phosphorus, which probably gives it its somewhat greater hardness, its higher limit of elasticity and its somewhat reduced ductility. Its elastic limit is found at 104 foot-pounds of torsion, or 31,200 pounds tensile resistance, and its ultimate strength is almost precisely that of the preceding specimen. Its elongation is 0.66 maximum. Unless more seriously affected by extreme cold than No. 98, it would be preferred for rails, and, perhaps, for most purposes.

No. 67 is a somewhat "higher" steel, made by the same process. It is less homogeneous than the two just examined, has greater strength and a higher elastic limit, but less ductility. Its resilience is very nearly the

same as that of Nos. 98 and 76. The elasticity of all of these steels seems very exactly the same. The ductility of No. 67 is measured by 0.40 elongation. At *d*, is seen another illustration of elevation of the elastic limit. The piece was left twenty-four hours under maximum stress. The torsional force was then removed entirely. On renewing it, as is seen, the resistance of the specimen was found increased in a marked degree.

No. 69 is an American Bessemer steel, containing not far from 0.5 per cent. carbon. The same effect is seen here that was before noted, an increase of hardness, a higher elastic limit, and greater strength, obtained however, by some sacrifice of both ductility and resilience. The elastic limit is approached at 130 foot-pounds of torsional moment, or 39,000 pounds tensile, and the maximum is 280 foot-pounds of moment and 84,000 pounds tensile resistance at 133°. Its maximum angle of torsion is 150°, its elongation 0.24.

No. 85 is a singular illustration of the effects of what is probably a peculiar modification of internal strain. It seems to have no characteristics in common with any other metal examined. Its diagram would seem to show a perfect homogeneousness as to strain, and a remarkable deficiency of homogeneity in structure. It begins to exhibit the indications of an elastic limit at *a*, under a torsional moment of 110 foot-pounds, or an apparent tensile stress of 33,000 pounds per square inch, and then rises at once by a beautifully regular curve, to very nearly its maximum at 16°, and 176 foot-pounds. The maximum is finally reached at 130°, and thence the line slowly falls until fracture takes place at 195°. The maximum resistance seems* to be very exactly 60,000 pounds to the square inch. Its maximum elongation for exterior fibres is about 0.23. The resilience taken at the elastic limit is far higher than with common iron, and it is seen that this metal, in many respects, may compete with steel. Its elasticity is seen to remain constant wherever taken. This singular specimen was a piece of "cold rolled" iron. It is probably really far from homogeneous as to strain, but its artificially produced strains are symmetrically distributed about its axis, and being rendered perfectly uniform throughout each of the concentric cylinders into which it may be conceived to be divided, the effect, so far as this test, or so far as its application as shafting, for example, is concerned, is that of perfect homogeneousness. The apparently great deficiency of homogeneousness in structure is readily explained by an examination of the pieces after

* With an exceptional case, of which this is an example, the scale for tension is incorrect. The tensile strength is probably higher than here given.

fracture; they are fibrous, and have a grain as thread-like as oak; their condition is precisely what is shown by the diagram, and the metal itself is as anomalous as its curve.

8. Tool Steels.—The "tool steels" differ chemically from the "low steels" in containing a higher percentage of carbon, and usually, in being very nearly, if not absolutely, free from all injurious elements. They are made in crucibles by melting down the blister steels which are the crude product of the process of cementation, or sometimes, by melting a charge composed of selected iron, a small proportion of manganese bearing alloy and the proper amount of carbon. Containing a higher proportion of carbon than the preceding class of metals, it is comparatively easy to secure homogeneousness by the introduction of manganese, and by the same means, to eliminate very perfectly the evil effects of any small proportion of sulphur that may be present. Their comparatively large admixture of carbon makes them harder, and reduces their ductility, and since the reduction of ductility occurs to a greater degree than the increase of strength, the effect is also to reduce their resilience. The working of these metals is more thorough than is that of the less valuable steels, or of iron. They are cast in comparatively small ingots, and are frequently drawn down under the hammer, instead of in the rolls, and are thus more completely freed from that form of irregularity in structure noticed so invariably in steels otherwise treated. The effect of increasing the proportion of carbon, is to confer upon iron the property of hardening, when heated to a high temperature, and suddenly cooling, and the invaluable property of "taking a temper," *i.e.*, of assuming, under proper treatment, any desired degree of hardness. The hard steels are, however, comparatively brittle, the hardening being secured at the expense of ductility. The effect produced upon the tenacity of unhardened steel, by increasing proportions of carbon is somewhat variable, since it is influenced greatly by the presence of other elements. For good steels unhardened, the writer has been accustomed to estimate tenacity by the following formula, which is approximately accurate, and may be often found useful

$$T = 60{,}000 + 70{,}000\ C.$$

in which T represents the tenacity in pounds per square inch, and C is the percentage of carbon contained in the metal. This subject will be considered at greater length after a series of experiments have been made to obtain more exact determinations.

Referring to Plate II, a set of diagrams will be found, having their

origin at 180°, which are *fac similes* of those automatically produced during experiments upon various kinds of tool steels.

No. 58 is an English metal, known in the market as "German crucible steel." It is remarkable as having a condition of internal strain which has distorted its diagram to such an extent as to completely hide the usual indication of the elastic limit. A careful inspection shows what may be taken for this point at about $14\frac{1}{2}°$ of torsion, when the twisting moment was about 120 foot-pounds, and the tensile resistance 36,000 pounds per square inch. The metal is homogeneous in structure, has an ultimate resistance of 302 foot-pounds of moment, or 90,600 pounds per square inch tensile resistance. Its resilience is evidently inferior to that of the softer metals, and also less than the next higher and better grades. This metal contains about 0.60 to 0.65 per cent. carbon. Its elongation amounts to 0.045.

No. 53 is an English "double shear steel," of evidently very excellent structure, but less strong and less resilient than the preceding. Its exterior fibres are drawn out three per cent.

Nos. 41 and 61 are two specimens of one of the best English tool steels in our market. The first was tested as cut from the bar, but the second was carefully annealed before the experiment. In this instance, annealing has caused a slight loss of resilience as well as a decided loss of strength. In No. 41, the limit of elasticity can hardly be detected, but seems to be at about the same point as in No. 61, at near 130 foot-pounds moment and 39,000 pounds tension. The ultimate strength is nearly 119,000 pounds per square inch. The proportion of carbon is very closely 1 per cent. Its section would reduce by tension, 0.05.

No. 70 is an American "spring steel," rather hard, but as shown by its considerable resilience, of excellent quality, resembling remarkably the tool steel No. 41. It differs from the latter, apparently by its much higher elastic limit. It is possible that this may have been caused by more rapidly cooling after leaving the rolls in which it was last worked. It is evident that, for exact comparison, all specimens should be either equally well annealed or should be tempered in a precisely similar manner, and to the same degree.

Nos. 71 and 82 are American tool steels, containing about 1.15 of carbon. The former is notable as having an elastic limit at 69,000 pounds, and a probable deficiency of manganese, producing the usual indication of heterogeneous structure. Both of these steels lack resilience, and are less well adapted for tools like cold chisels, rock drills,

and others which are subjected to blows, than for machine tools. They have a maximum elongation, respectively, of but 0.013 and 0.03.

Interesting and instructive as the study of these curves may be made, the information obtained from them is supplemented, in a most valuable manner, by that obtained by the inspection of the fractured specimens, upon which the peculiar action of a torsional strain has produced an effect in revealing the structure and quality of the metal that could be obtained in no other way.

Fig. 8 represents the appearance of No. 68, and Fig. 9 that of No. 58, while the peculiarities of the finest tool steels are seen in No. 71, as shown in Fig. 10. The smooth exterior of No. 68, which is a companion specimen to that giving diagram 69, and its bright and characteristic fracture, resembling that of No. 22 somewhat, together indicate its nature perfectly, the first feature proving its strength and uniformity of structure, and the second showing, even to the inexperienced eye, its toughness. This is a representative specimen of low steels. No. 58 is seen to have retained, even more than No. 68, its original smoothly polished surface. Its fracture is less waxy, and much more irregular and sharply angular. The crack running down the side of the neck shows its relationship to the shear steels which much oftener exhibit this effect of strain, in consequence of their lamellar character. No. 58 is evidently intermediate in its character between the soft steels, like No. 68, and the tools steels which are

Fig. 8.

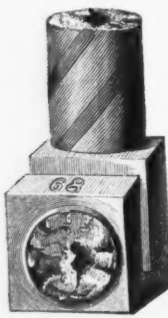


Fig. 9.

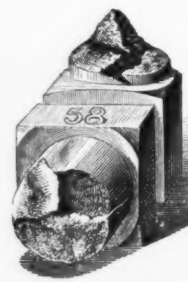


Fig. 10.

represented by No. 71, Fig. 10. In this test-piece, the fracture is ragged and splintery, and the separated surfaces have a beautifully fine, even grain, which proves the excellence of the material. The surface which was turned and polished in bringing the metal to size remains as perfect as before the specimen was broken. By an inspection of the broken test pieces in this manner, the grade of the steel, and such properties also as are not revealed by an examination of the diagram of strain, are very exactly ascertained by a novice, and to the practical eye, the slightest possible variations of character are readily distinguishable.

9. CAST-IRON.—The diagrams of strain having their commencement at 100°, have been obtained from cast-iron and from malleableized cast-iron.

Nos. 23 and 24 are those given by a good dark grey foundry iron from Pennsylvania. No. 25 represents the curve of light grey scrap, and No. 30 is from a very fine white Lake Superior charcoal iron. The latter is seen to be exceedingly hard and rigid, the resistance of the piece rising very precisely in proportion to the angle of torsion until it snaps at last under a moment of over 200 foot-pounds, equivalent to a tension of 60,000 pounds per square inch, and with a maximum elongation of one-tenth of one per cent. This is a most extraordinary resistance, but it is evident that, notwithstanding its immense strength, this material would be valueless for ordinary purposes in consequence of its excessive brittleness. When the torsional effort had reached about one-half its maximum amount the piece was released. The pencil retreated along a nearly vertical line *e*, which it again traversed as the strain was gradually renewed. Here as in many other cases, where a similar experiment was made, evidence is given of the truth of the statement originally made by Hodgkinson,* that every load produces a set. As will be shown, subsequently, however, it is not true in perfectly homogeneous bodies free from strain, and within their elastic limit. The light grey iron has a limit of elasticity at near one-half the maximum reached by the white iron, without any sign of reaching the limit of its elasticity. The grey has more ductility than the white iron, but has only about two-thirds the resilience of the latter. The dark grey irons are evidently better than either of the lighter grades, except in power of carrying an absolutely static load. The actual stretch of the outer surface particles is very nearly the same in all three. They are excellent specimens of their class, and considerably better than ordinary irons.

* Reports of British Association; also Civil Engineer and Architects' Journal.

No. 37 is a "malleableized cast-iron," made from the extraordinary metal illustrated in No. 30. The process of malleableizing consists in decarbonization by heating the casting made from good white iron, in contact with iron oxide or other decarbonizing material. Without removing any other constituent than the carbon, it produces a crude steel or an impure wrought-iron. When performed in the usual manner, melting the cast-iron in a cupola in contact with the fuel, and with some flux, and then carrying the process of malleableizing to the usual extent, a metal is obtained such as is illustrated by the diagram marked 37. It retains the strength of the cast-iron, and acquires some ductility.

No. 30 yielded 7° before fracture, while No. 37, vastly more ductile and resilient, only broke after a torsion of 39°, and a maximum elongation of 2 per cent. Taking the precaution to melt the iron in an "air furnace"—a form of "reverberatory"—and conducting the process of malleableizing more carefully, a still more valuable material was obtained.

No. 35 represents this iron. Its resemblance to wrought-iron, both in appearance of fracture and in its strength and ductility, are greatly increased. It has a high limit of elasticity—over 20,000 pounds per square inch—and such ductility that it only breaks after a torsion of nearly 168°, and an elongation of "fibre" of 0.35. It is not very homogeneous, but it is as strong, and almost as tough, as a good wrought-iron. This material has especial value for many purposes, because of the facility with which awkwardly shaped pieces can be made of it. In many cases, it will prove as good as wrought-iron and far cheaper.

Fig. 11.

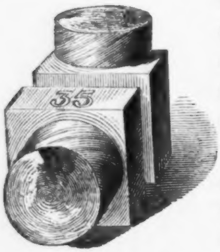
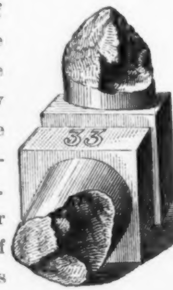


Fig. 11 shows the appearance of this last specimen. Its resemblance to wrought iron is very noticeable. The lines running like the thread of a screw around the exterior of the neck, and the smooth even fracture in a plane precisely perpendicular to the axis, are the instructive features. Fig. 12, representing No. 33, is a specimen similar in character to No. 37. The comparative lack of ductility, its less regular structure, and its less perfect transformation are perfectly exhibited. Fig. 13 is an excellent

Fig. 12.



ent of the white iron as cast and without malleableizing. Its surface where fractured, has the general appearance of broken tool steel. The color and texture of the metal are distinctive, however. It has none of the "steely grain." Fig. 14 represents the dark grey iron. Its color, its granular structure and coarse grain are markedly characteristic and no one can fail to perceive, in the specimen, the general character which is exactly given by the autographic diagrams of the testing machine.

Fig. 13.

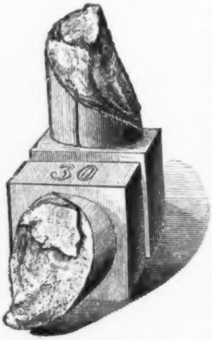
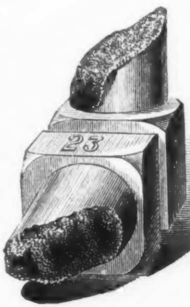


Fig. 14.



10. Other Metals.—The diagrams numbered 87, 88 and 89, are those of copper, tin and zinc. These specimens are all of cast metal, carefully selected under the direction of the writer and moulded and cast at the Stevens Institute of Technology. They exhibit neatly the wonderful superiority which the various kinds of iron and steel possess over the other useful metals. These metals all take a set under very small strains, pass their limits of elasticity at some indeterminable, but evidently low point, and possess very slight tenacity.

Zinc, No. 89, by the regularity of its curve shows a very uniform structure. It increases very gradually in resistance to torsion, until it reaches the angle 50°, at which point it has a moment of torsional resistance of 36 foot-pounds, and a maximum tenacity of about 10,800 pounds per square inch. It loses its power of resistance, after rupture commences, as regularly, but not as slowly, as it acquired it, and rupture becomes complete at 63°. Its resistance is exceedingly small, and it is evidently unfit to bear either static or dynamic force. Its stretching power has a maximum of 0.04.

Tin, No. 88, is equally remarkable for its exceedingly feeble resistance and its great ductility. The specimen was excellent, both in quality of metal and in closeness of structure, as was indicated by the clearness of the "tin cry" heard while undergoing the test and by the fine, smooth, clean fracture. The character of the curve is similar to that of zinc, but has far greater extent. Its elastic limit is quite indeterminable. The

outline of the diagram indicates very perfect homogeneousness. The maximum resistance to torsion is found at 240°, and under a stress of 19 foot-pounds. Its tenacity deduced from the diagram is, at most, but 5,700 pounds per square inch. Rupture occurs very gradually, and the piece separated entirely at 355°. Notwithstanding its great ductility, its low tenacity produces a low resilience, although in this quality it excels zinc, which latter metal had nearly double its strength. Its elongation by tension would have reduced its section to 0.6 of the original cross area, if that reduction were proportional to the ductility shown by the diagram.

Copper, No. 87, cast in green sand, like the zinc and tin just described, was found, on examination of the fracture, to differ from them in being exceedingly porous. The effect of this fault has been to weaken it seriously. Its curve closely resembles that of zinc, but is abruptly terminated by the piece suddenly breaking off at 46°. It reaches a maximum sooner than zinc, at 29°, and its greatest resistance to torsion is 36 foot-pounds, or to tension 10,800 pounds per square inch, precisely the same as zinc. Its ductility has a value of one and a half per cent. Its resilience is somewhat less than that of zinc. Its limit of elasticity is difficult to determine, but has been taken at 1½° where the moment of resistance is 13 foot-pounds, equivalent very nearly to 3,900 pounds tenacity, per square inch.

No. 134 is the curve of cast copper, precisely similar to No. 87, but cast in a dry sand mould. The marked difference between these specimens is probably due, not only to the difference in degree of porosity which arises from the presence of vapor, which permeates the casting in one case, filling it with bubble holes, and which is almost unobservable in the last, but the slower cooling of the dry sand casting also probably produces its effect in strengthening the metal. This last specimen has a limit of elasticity at not far from 13¾°, and under a torsional stress equivalent to a tension of 5,400 pounds per square inch. The maximum values of these quantities are found at 21°, and are 42 foot-pounds, and 12,600 pounds per square inch respectively. The resilience of the specimen is much greater than that of the preceding, and its maximum elongation is .026. Altogether, this is far better than the preceding, and it would seem that copper, and probably all its alloys, should, when possible, be cast in dry sand, to secure density and strength.

No. 141 is a piece of forged copper, hammered into a one-inch square bar, from a piece originally 3½ inches wide and ⅞ inch thick. The most striking property noticed is its immense ductility, far exceeding that of any other piece of metal yet tested, and, in amount, many times as great

as the cast metal. Its limit of elasticity is reached very quickly, although it is impossible to say precisely where it occurs. Comparing its "elastic line" with the initial portion of the curve, it is seen that the slightest force produces a set which is proportionally large as compared with the sets of other metals. The curve rises very regularly and gradually to a maximum, which is only attained, however, after a total angle of tension of 450°, and which measures 96 foot-pounds moment, or 28,800 pounds per square inch. Rupture is finally obtained after a torsion of 543°. The maximum elongation is 210 per cent., the most elongated lines of particles being finally left of 3.100 times their original length. Had this change of form occurred by reduction of section, the fractured area would have been but .323 the area of original section. The resilience of this piece of metal is evidently insignificant within the limit of which it would be seriously distorted by a blow, but is quite large in amount where resistance extends to the point of rupture. This is perfectly consonant with that knowledge of the material which every mechanic derives from experience with it. Here, however, we have a complete account of its properties, written out by the material itself with definite and accurate measures.

11. General Conclusions.—These plates, exhibiting the diagrams, which are the autographs of all the useful metals, illustrate sufficiently well the remarkable fullness and accuracy with which their properties may be graphically represented, and the convenience with which they may be studied, with the aid of so simple a recording machine. A comparison of results deduced as shown, with those obtained, so far as they can be obtained at all, the usual method of simply pulling the specimens asunder, and trusting to, sometimes, unskillful hands and an untrained observer, for the adjustment of weights and the registry of results, will indicate the close approximation of this method in even ascertaining the behavior of the metal in tension. On examining the beautifully plotted curves given by Knut Styffe, as representing the results of the experiments, made by him and by his colleagues, with a tensile machine, no one can fail to be struck with the similarity of those diagrams to the curves here produced automatically, and it will be readily believed that not only must there be very perfect correspondence of results where the two methods are carefully compared, but, also, that any theory of rupture must be defective which does not apply to both cases. The equations of the curves here given and those of the curves obtained by Styffe must have forms as similar as the curves themselves.

The constant ratio here assumed between the torsional resistance and

the tensile strength of the metals, and of homogeneous materials generally is based upon a comparison of the results here given with those obtained from the irons by tensile test, by the writer, and is confirmed by a compilation of results given by other experiments on the same brands.

12. Testing within the limit of Elasticity.—In determining the value of materials of construction, it is usually more necessary to determine the position of the limit of elasticity and the behavior of the metal within that limit than to ascertain ultimate strength or except, perhaps, for machinery, even the resilience. It is becoming well recognized by engineers who are known to stand highest in the profession, that it should be possible to test every piece of material which goes into an important structure and *to then use it* with confidence that it has been absolutely proven to be capable of carrying its load with a sufficient and known margin of safety. It has quite recently become a common practice to test rods to a limit of strain determined by specification, and to compel their rejection when found to take a considerable permanent set under that strain. The method here described allows of this practice with perfect safety. The limit of elasticity occurs within the first two or three degrees, and, as seen, the specimen may be twisted a hundred, or even sometimes two hundred times as far without even reaching its maximum of resistance, and often far more than this before actual fracture commences. It is perfectly safe, therefore, to test, for example, a bridge rod up to the elastic limit, and then to place the rod in the structure, with a certainty that its capacity for bearing strain without injury has been determined and that formerly existing internal strain has been relieved. The autographic record of the test would be filed away, and could, at any time, be produced in court and submitted as evidence—like the "indicator card" of a steam engine—should any question arise as to the liability of the builder for any subsequent accident, or as to the good faith displayed in fulfilling the terms of his contract. A special machine has been designed for this case.

13. The above will be sufficient to show the use and the value of this method. In the course of experiment upon a large number of specimens of all kinds of useful metals and of alloys, a number of interesting and instructive researches have been pursued, and some unexpected discoveries have been made. Before taking up the theory of rupture, the construction of equations, and the determination of their constants, a section will be devoted to an account of these investigations.

PLATE I.

AUTOGRAPHIC STRAIN-DIAGRAMS OF WOODS

PRODUCED BY THE

TESTING MACHINE OF PROFESSOR R. H. THURSTON.

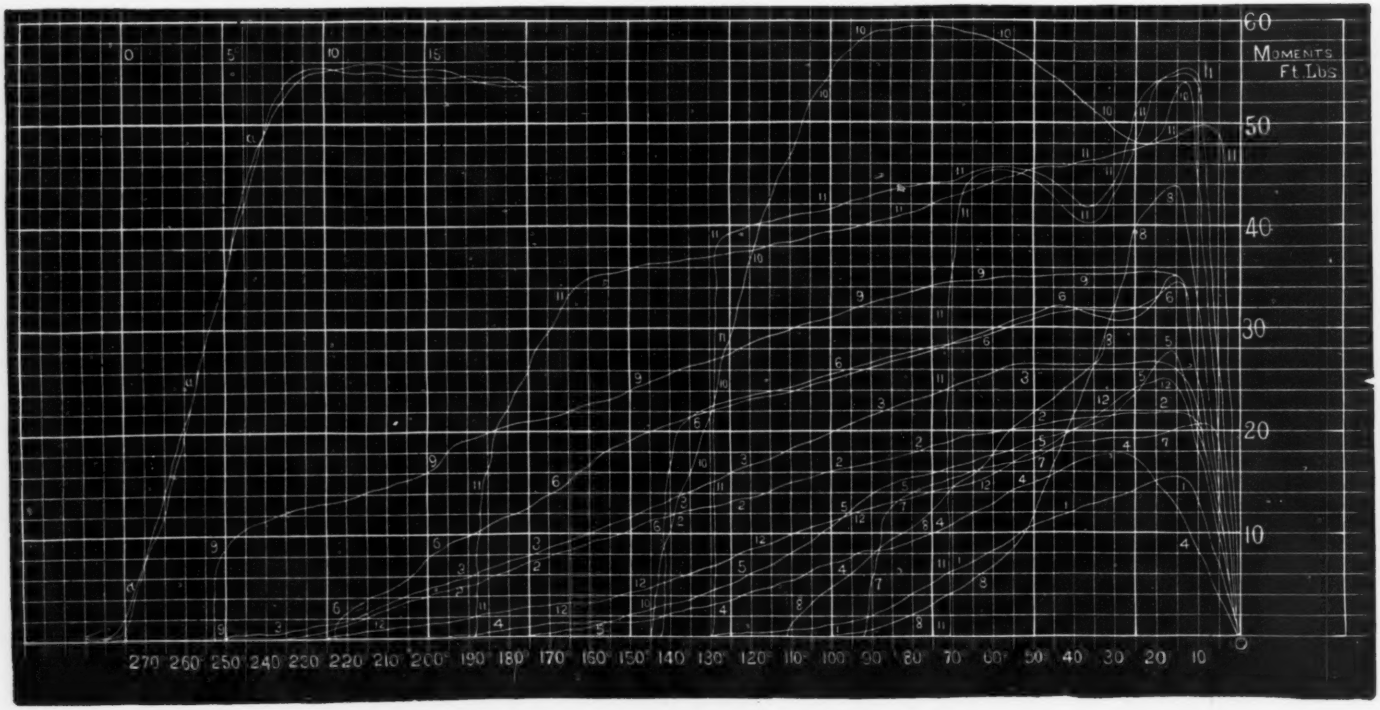


PLATE II.

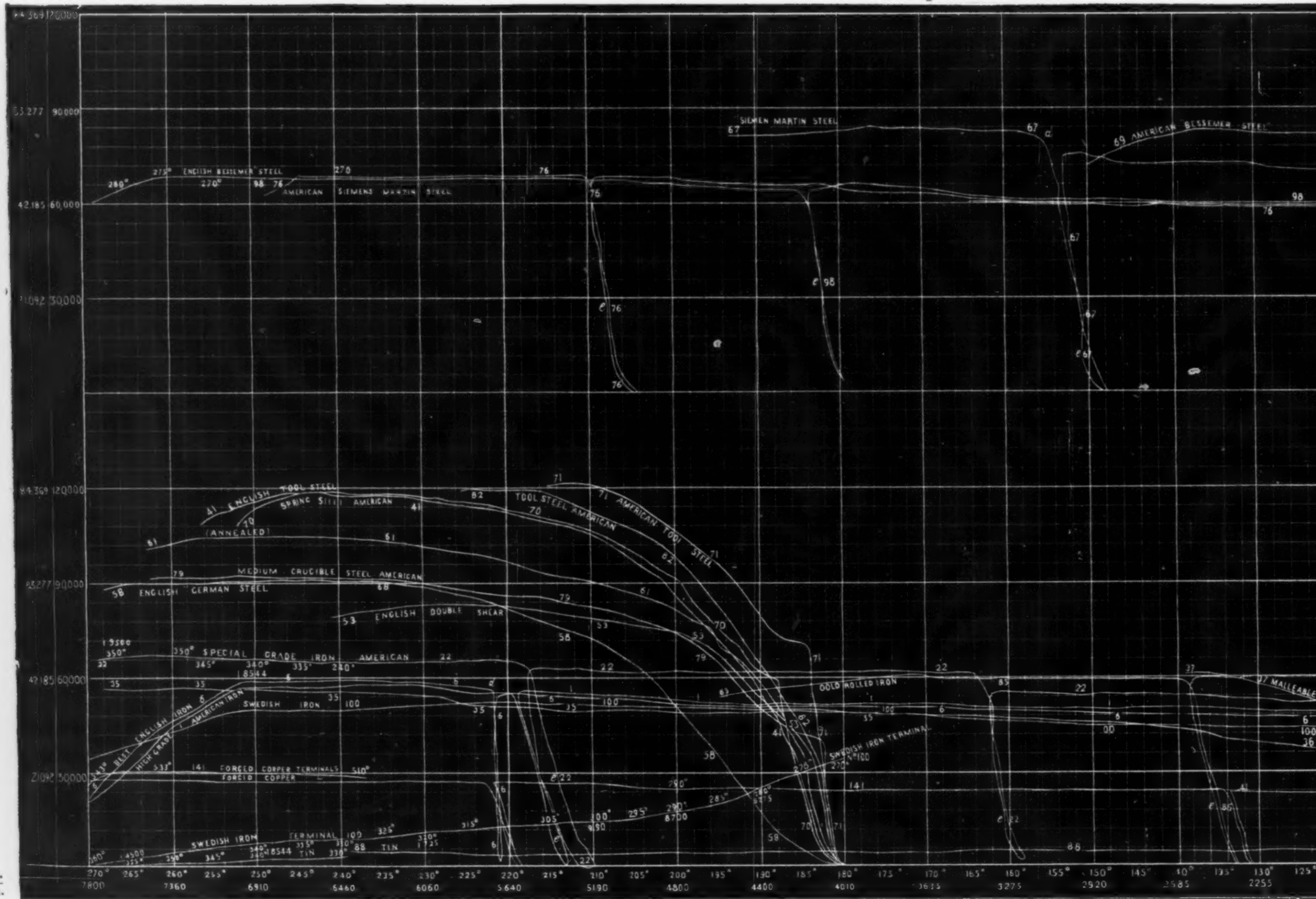
AUTOGRAPHIC STRAIN-DIA

PRODUCED BY THE

TESTING MACHINE OF PROFESSOR

APPROXIMATE
TENSILE RESISTANCE.

Kilogrammes per square Millimetre. Pounds per square Inch of Section.



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I.

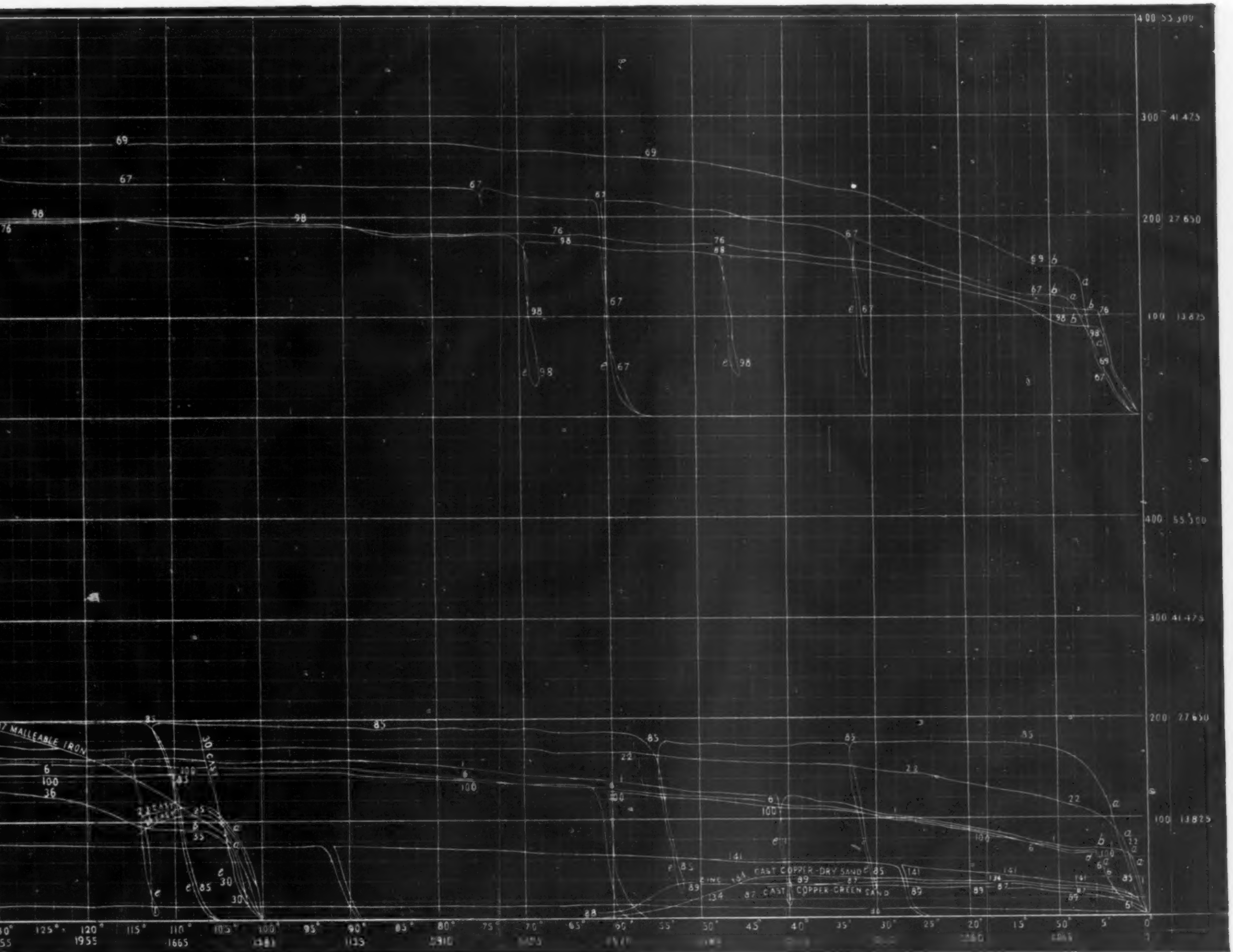
IAGRAMS OF METALS

THE

SSOR R. H. THURSTON.

Torsional Moments.

Foot-Pounds. | Kilogram-metres.



Angle of Torsion.

Elongation.

LXXVII.

CENTRAL AVENUE BRIDGE, AT NEWARK, NEW JERSEY.

A Paper by ALFRED P. BOLLER, C. E., Member of the Society.

PRESENTED JANUARY 5TH, 1874.

The following account of the "Central Avenue Bridge," with accompanying drawings, describes a novelty in bridge engineering, not because of any wonderful constructive features, but rather from the peculiar requirements of the case.

Central avenue is one of the main new avenues leading out of the city of Newark, and the commissioners under whom it was laid out were evidently impressed by the fact that a "straight line is the shortest distance between two points." The Morris Canal follows the crest of the hill west of Newark, until a little north of the avenue, where it takes a turn to the head of the plane, leading through the city, on its way to the Passaic river. Central avenue is intersected by this turn in the canal at such an angle, as is shown on the map, that its bounding lines at the crossing, measure 270 feet on the south side, and 113 feet on the north. The avenue being 80 feet wide, one side is some 50 feet in advance of the other. In addition to this, two streets intersect the avenue, as shown, immediately over the canal.

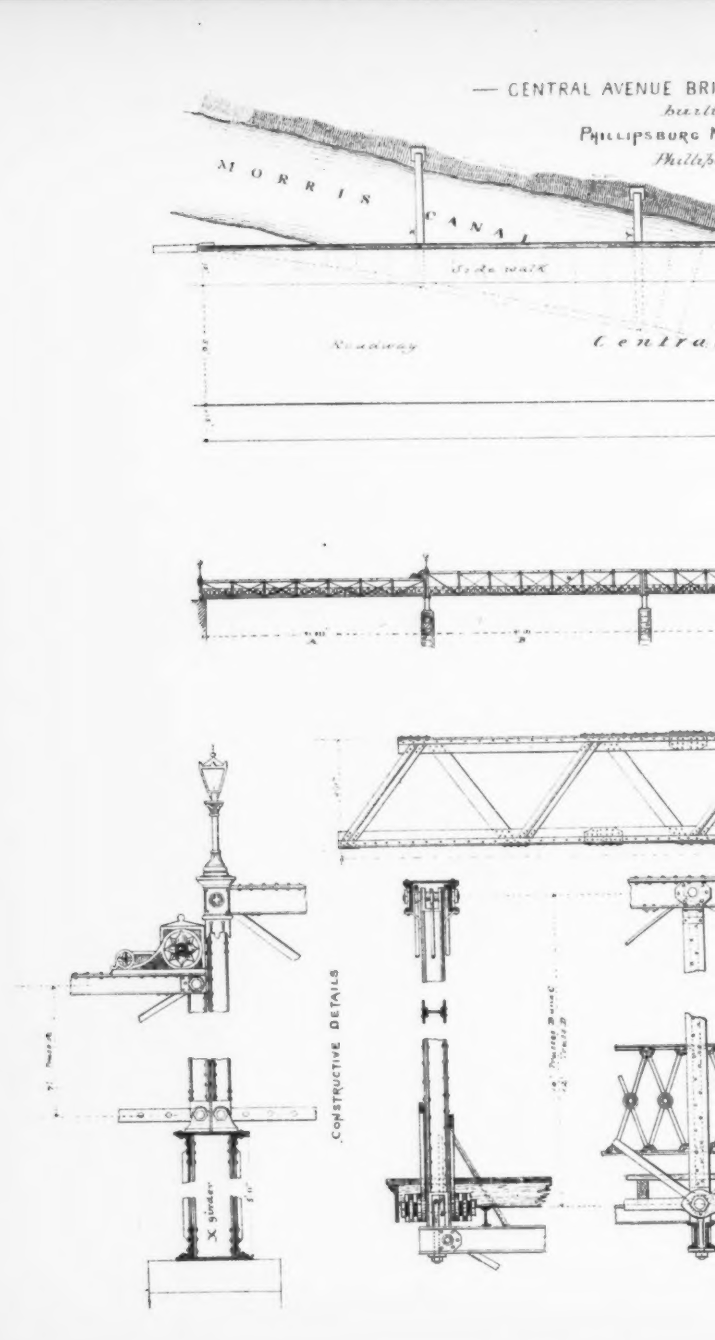
The canal authorities would not permit any pier or other support to be in the canal, or any portion of the structure that should be built to sustain these streets, to approach the tow-path nearer than 10 feet. The problem, then, was to bridge the canal in such a way as to give an unbroken floor for the full area of the streets, and to sustain it by trusses which would not encroach upon the canal. Two modes were considered —one to span the distances given above, with two standing trusses, and the other, to throw across, on the longer side, heavy girders from the retaining walls or abutments, to masonry piers on the berme side of the canal, using the girders as piers, to form shorter spans.

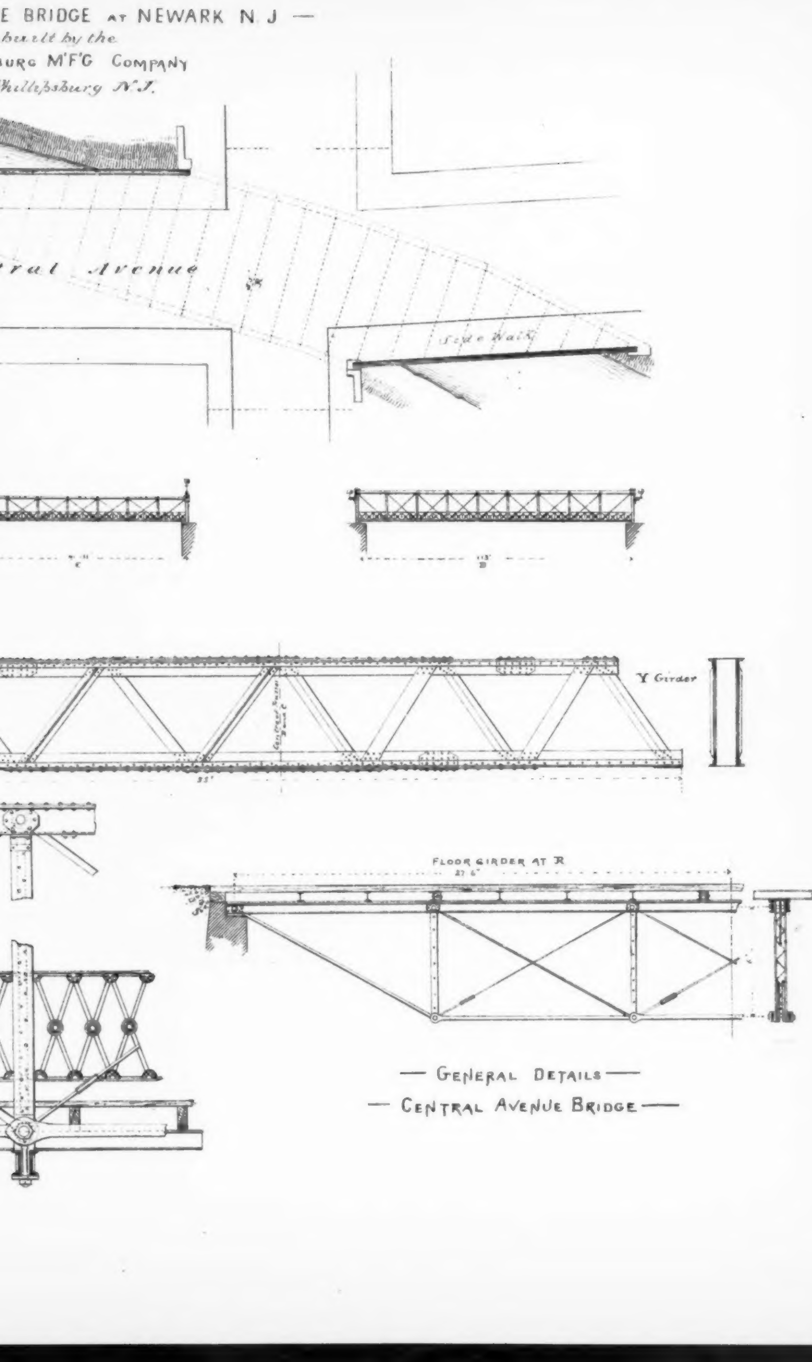
The objections to the first consisted in the expense attending it on

the longer side, and the difficulty of arranging sufficient room for a proper breadth of end-bearings, to say nothing of the fears of the commissioners regarding the stability of the long standing-truss which must depend almost entirely upon "outrigger bracing" from the floor-beams. By the advice of Mr. James Owen (Member of the Society), the county engineer, and under whose supervision the bridge was erected, the committee of the Board of Freeholders concluded to adopt the second plan—that presented by the Phillipsburg Manufacturing Company, who were awarded the contract.

The bridge was finished in January, 1874; its construction is clearly shown on the drawings. The masonry walls on the sides of the canal are shown in dotted parallel lines, as are also the floor-girders, the largest of which, 55 feet in length, is about the "square" distance across the canal. One of these girders is marked *R*, and shown in detail. There being no floor-girders parallel, excepting a few short ones, supported by trusses, *A* and *B*, the iron stringers, to support the 3-inch yellow pine plank, could not be rolled until all the girders were exactly placed and the lengths measured from the executed work. These stringers are 6-inch rolled beams, placed about 3 feet apart under the roadway, and about 4 feet apart under the sidewalks, the sidewalks being raised some 8 inches above the roadway. The girders forming the piers are 55 feet in length, and are plate and angle-iron, double Warren girders, 2 feet wide; the heavier one is shown. The main trusses are entirely of wrought-iron, and are constructed on the pin-connection quadrangular system, in which all parts are accessible to painting and inspection, as is, in fact, the whole of the iron-work. The railing, of wrought-iron, is in panel lengths, and placed between each pair of the posts of the trusses.

It is believed that, with this brief description, the drawings are sufficiently intelligible to those interested in constructions of this character.


— CENTRAL AVENUE BRI
PHILLIPSBURG N
MORRIS
CANAL
Side walk
Roadway
CONSTRUCTIVE DETAILS
X girder



— GENERAL DETAILS —
— CENTRAL AVENUE BRIDGE —

LXXVIII.

THE ELEMENTS OF COST OF RAILROAD FREIGHT TRAFFIC.

A Paper by O. Chanute, C. E., Member of the Society.

(Prepared from notes used in the discussion of "The Elements of Cost of Railroad Traffic.")

At the Evening Meeting—February 18, 1874.

The question of passenger traffic is so completely treated in the very able paper of Mr. A. Fink, read this evening, that I shall confine my remarks to that of freight business.

The subject of determining the cost of railroad transportation is seemingly a very simple one. So simple, that to one unacquainted with the subject, it doubtless appears capable of easy solution, after a brief investigation. And, indeed, when at the close of the year, the report of a railway company is published, it is comparatively an easy matter to reduce its whole business to an average number of passengers and of tons of freight transported one mile, and to ascertain approximately the average charges and cost of each. We thus obtain, however, only averages and nothing more. It is when we attempt to look further into the subject, and enquire why this average cost varies upon different lines, and what are its various elements, or to ascertain what profit has been made upon a particular class of the traffic, and what portion has been done at a loss, that we find very considerable intricacy and complication.

It is found, not only that this cost varies greatly upon different roads, in consequence, as we shall see, of local peculiarities, but that even upon the same road, it varies materially from year to year and from month to month. A comparison, by the reader, of the monthly earnings and expenses of almost any railroad, will exhibit the fact that their ratio is far from being uniform; that the cost of carrying a passenger or a ton a mile varies 30 and 40 per cent. between the different months, and that in some cases the whole of the traffic is actually worked at a loss during some portion of the winter months.

The ultimate purpose of the investigation, which led to the remarks from which these notes are taken, being to ascertain the cost of particular operations upon a single road, it was thought necessary first to ascertain

whether the division of the various elements of cost had been assumed correctly, by comparison of several roads. The present object, however, being only to indicate what those elements are, and the manner in which they burden the traffic differently upon the several roads; the present paper will be confined to a discussion of those elements, to the pointing out of their numerous combinations, and to such general deductions as seem to spring therefrom.

And first it may be stated that the problem of separating the various elements of the cost of railroad traffic, is probably incapable of exact solution with our present knowledge. The various expenses of which they are composed are combined with each other in so many different operations, that it is very difficult to separate them, so as to ascertain the cost of any particular class of traffic with mathematical certainty. We may approximate to it, however, and thereby gain clearer ideas on a subject which we will find to be very intricate.

Even the preliminary step of separating the cost of passenger from that of freight traffic is found difficult and unsatisfactory. Not only are many services incidental to each performed by the same agents, and there is great uncertainty as to what proportion of the maintenance of the way and works should be charged to each, but there is great diversity in the mode of keeping the accounts on different roads, and the basis furnished by the companies themselves is found upon examination to be arbitrary and somewhat unreliable. For the present, however, the figures have been taken as returned, merely making such corrections as seemed obvious, and the apportionment will be made from this.

Indeed, so complicated does the subject become, upon even a slight examination, that railway managers are very loath to commit themselves to positive statements as to the cost of particular portions of their traffic, and much of the reticence, which has recently been sharply criticised during the discussion which has been going on in the public prints on this subject, is no doubt due more to an unwillingness to put themselves on record concerning many points which are by no means clear even to themselves, rather than to any disinclination to enlighten the public.

Table I, prepared from the Reports of the Engineer of the State of New York, shows the variations of charges made, and the cost per ton per mile, upon seven of the railroads of this State, during the last ten years, as well as the varying ratio of the cost of operating to the total expenses. It exhibits not only that the cost has been four times as great on some roads as upon others, but also that it has materially varied

from time to time upon the same road ; so that, for instance, had the average rates of charges prevailing in 1863 been maintained until 1865, when the war had raised the cost of labor and materials, six out of the seven roads would have been operated at a loss.

In order to account, if possible, for the cause of the difference of the cost per ton per mile, upon these several roads, an analysis has been made of their expenses for the year ending on the 30th of September, 1872. We thus obtain, it is true, only averages resulting from many different operations for each road, but we may be able to draw some inferences from them. Table II shows the cost of transporting freight for 1872 on those roads, under seven general elements of cost, reduced to four common standards (such as could be obtained from the reports), under the heads of cost per ton per mile, cost per mile operated reduced to equivalent single track, cost per mile run by freight trains, and cost per ton transported. If, upon examination, there appears to be any uniformity in any one element of cost, under any one head of the division, it would be considered probable that this element operated alike on the different roads, and that the particular head under which the coincidence occurred was probably the best common measure of comparison. It will be found, however, that the resulting burden imposed by each upon the traffic, differs widely upon the different roads.

The division of the elements of cost, adopted in the classification, is as follows :

1.—That which is here termed "Roadway charges," and which consists of the repairs and renewals of the earth works, masonry, ballast and wooden portions of the roadway—such as cross ties, bridges, buildings, fences, &c. The repairs of earthworks, masonry, ballast and cross-ties, have been obtained by distributing under this head one-third of the amounts charged to "Repairs of roadbed, except cost of rails." Taxes are also included in this account, and it may fairly be considered as a fixed ratable charge, independent either of the volume or character of the traffic ; representing in a great measure the wear and deterioration from the action of the elements, and proving more or less of a burden, per ton per mile, in proportion to the business.

2.—The "General expenses" comprising the expenses of general management and incidental contingencies. To be strictly accurate, this account should also include the cost of soliciting for and obtaining business, but as this cannot readily be separated from the amounts reported for "Agents and clerks," the whole of the "Contingencies" account, has

been included as an offset. This varies somewhat in proportion to the volume of the trade obtained, as well as with the length of the road, but seems quite independent of the distance the traffic is to be carried. It may in most cases be considered as an arbitrary charge of so many cents per ton obtained.

3. "Station service," comprising the items termed "office expenses," "agents and clerks," "loading and unloading," and "watchmen and switchmen," in the official reports, and covering the cost of handling and billing the freight. This varies nearly in proportion to the tonnage handled, but is quite independent of the distance it may be carried over the road. It may be considered as a specific or arbitrary charge per ton, to be pro-rated over the number of miles the goods are conveyed.

4. "Track repairs," which includes the surfacing of the track, and the repairs and renewals of the rails, spikes and joint fastenings. This varies in some measure, but not in direct ratio with the tonnage transported, and the speed at which it is carried. It is greatly affected by the character of equipment placed upon the road, as well as by the permanence of its construction, and its peculiarities of climate and soil. In the absence of more accurate information on this subject, its best measure is probably the miles run by trains.

5. "Car service," embracing the lubrication, repairs and renewals of freight cars. This varies most nearly with the mileage made by the cars, but the time required to make a trip, unload the car, and return it to the general service, becomes an important component, and seriously increases the cost of local traffic.

6. "Train service" may be said almost to alone represent the transportation proper. It consists of the wages of the "conductors and train men," "enginemen and firemen," "fuel," lubrication, water service, and the repairs and renewals of the locomotives. This alone varies both with the tonnage and the distance it is carried, and alone can correctly be compared for different roads by reduction to tons transported one mile. Its cost upon each is affected by the character of the gradients and curves, which limit the maximum train which can be taken over the line, the proportion of empty cars which must be hauled in consequence of the preponderance of tonnage in one direction, the cost of wages and fuel, and in the case of new roads, by the impossibility of securing full loads at all times for the trains which it is desirable to run regularly.

7. The "Insurance," which mainly consists of the loss and damage

accounts, and which is dependent upon the value or perishable quality of the articles carried.

Having thus distributed the expenses according to these seven different elements, and constructed the comparative tables, we are enabled to take up each division in detail, and by comparing the resulting charge per ton per mile as deduced from specific expenditures, whether it be referred to the standard of cost per mile operated, or per mile run by freight trains, or per ton transported, to enquire in what manner, and to what extent each element forms a charge upon the general traffic, and helps to swell the cost of the whole. This analysis can only be followed by a close and frequent inspection of the table, and although, no doubt, irksome to the general reader, is yet necessary to an understanding of the very complex relations which the expenses bear to each other.

An examination of the table thus constructed, at once discloses differences in cost, which no possible theory as to relative economy or efficiency of management can account for. It is seen not only that like operating expenditures upon different roads, whether referred to cost per lineal mile or per mile run by freight trains, impose very unlike burdens upon their aggregate traffic per ton per mile, but also that the character of that traffic makes a very great difference in the cost of transacting it.

Thus, the Rome, Watertown & Ogdensburgh R. R., which has spent but $2,670.70 per mile operated, upon its freight business, shows a cost of 2.641 cents per ton per mile, while the Erie Railway, which has expended $8,212.58 per mile, or over three times as much, shows but a cost of 1.037 cents per ton per mile, or less than one-half as much. The Rensselaer and Saratoga R. R., which has run its trains for $1.42 per mile, yet finds the cost 2.306 cents per ton per mile, while the New York Central R. R., which has run its trains for $1.35 per mile, finds the cost but 1.043 cents per ton per mile.

These differences illustrate the influence on cost, of the total volume of business done. It is seen at once that some elements of cost are in the nature of fixed charges, and neither materially increase nor diminish, whether a large or a small business be done. That others again are specific or arbitrary charges, which are nearly constant, whether the traffic is to be conveyed a long or a short distance; and again, others increase with the business, but not in direct ratio to it; while of those which increase in strict proportion per ton per mile, the cost is probably not over one-third of the whole. One effect of this is to burden those roads which do a small business with a much higher cost than those which

have developed a large traffic. There are certain expenses which must be incurred to keep the road running, and they prove less or more onerous, as the tonnage is large or small.

More important still as affecting the cost, is the character of the traffic. The road making the best showing in the table is the Syracuse, Binghamton & New York, The cost for 1872 was but 0.704 cents per ton per mile, and by reference to Table I, it will be seen that it transported freight for two years, at a cost of 0.55 per ton per mile. Yet this road has expended this year but $3,273.74 per mile of road, while the New York & Harlem R. R., which has expended but $4,531.88 per mile, exhibits a cost five times as great, or 3.635 cents per ton per mile. The New York Central, on the other hand, has expended $8,127.72 per mile operated, and its cost is 1.043 cents per ton per mile. So that we see that it is not alone the doing of a large business which cheapens the cost, by distributing the fixed charges over a greater number of tons, but also the character of that business which may require more or less looking after or incidental expense.

Comparing more in detail the Syracuse, Binghamton & New York R. R., and the New York & Harlem R. R., which exhibit the two extremes in cost per ton per mile, we note that certain of their expenses per mile operated are substantially the same. The roadway charges, the track repairs, and the insurance, practically agree. They aggregate $1,731.74 per mile, in the case of the former road, and $1,662.15 per mile in the case of the latter; yet they impose exceedingly unlike burdens per ton per mile. It is in the other elements that the great saving occurs on the S., B. & N. Y. R. R. The general expenses are only one-quarter as much per mile operated, and less than one-fifteenth as much per ton per mile as on the Harlem R. R. The station service is less than one-half as much per mile, and imposes a burden only one-tenth as great. The car service and train service are only about one-half as much per mile, and become a charge of only about one-sixth as much per ton per mile, so that those expenditures which are specifically the same on these two roads, not only differ greatly in the result per ton per mile, but those expenses which are variable, are much greater in the one case than in the other. The explanation is, not only that the one has a larger, and the other a smaller relative traffic, but that the former road does a through, and the latter a local business; and that the first mainly transports coal, and the second merchandise. It will be well, therefore, to examine each element of the cost separately, and to enquire how and to what extent it burdens the traffic.

First, as to the "roadway charges," we notice a remarkable uniformity per mile of road operated, in the first five of the seven roads in the table. They have each expended about $800 a lineal mile, in replacing perishable material, and maintaining their works, and yet this cost has charged their traffic from one-tenth to six-tenths of a cent, per ton per mile in proportion to the greater or less volume of business done. On the two last roads, which have expended less than the others (perhaps because it was not necessary that they should fully make the wear good for that year), the burden per ton per mile is, nevertheless, about three times as great as upon the road which has expended the most. Although part of the wear of wooden structures is due to the action of the trains, yet the greater part of it is caused by exposure to the weather. As this proceeds slowly, it is not necessary to renew an equal portion every year, and the amount expended will vary on each road in different years; but it will be noticed from Table I, that the yearly cost per ton per mile varies most widely upon the roads with the lightest business, probably in consequence of periodical renewals.

It may here be stated, that since making up Table II., the writer has had reason to believe that the proportion adopted of one-third of "repairs of roadbed" for the roadway charges, is somewhat in excess of the truth, and that a sum of about $600 or $700 a year a mile, would probably cover the general cost of this element; but whether $600 or $800 per mile of road, the amount is a fixed charge, in no way affected by the volume of business, but burdening it more or less, accordingly as it is large or small, and therefore very onerous to new roads with light traffic. These may perhaps evade the charge for a time, while everything is new, but the wear and deterioration is constantly going on, and sure some day to call for fresh expenditures, either from current earnings or from capital.

The burden imposed by "general expenses" is partly fixed and partly arbitrary. That is, there are some expenses which must necessarily be incurred in managing the line, and some which increase with the business, but not in proportion to it. As we should expect, therefore, the table exhibits great variations per ton per mile, while there is some correspondence in the columns in which it is given per mile operated and per ton transported. It is seen also to be affected by the character of the traffic, in the case of the Syracuse, Binghamton & New York R. R., whose business evidently requires very little soliciting or managing expense. Although its percentage to the other operating

expenses is small, it may form, in the case of the long thin lines recently built in the West, an important element of the cost of the light traffic which they may expect for some time.

The "station service" imposes a charge which is much the same, whether the article goes a short or a long distance over the road. For four out of the seven roads, it is seen to amount to about 32 cents per ton transported, irrespective of distance ; and a table will hereafter be given to illustrate how this charge alone may cause the cost to vary from 4 cents a ton a mile, to about half a cent a ton a mile. In fact, it is clear that the cost is the same for loading and unloading, checking and billing the freight, whether it is transported over the road 10 miles or 1,000.

We notice, however, some anomalies in the table. Thus station service on the Syracuse, Binghamton & New York R. R., is but 4 cents a ton transported ; while on the New York & Harlem R. R. it is 35 cents a ton. This is explained by the fact, that while the cost per ton per mile, and per ton transported, is made up from the total number of tons carried, yet a part, in some cases a very considerable part, is received or delivered loaded on the cars, either in the interchange of business with other roads, or as being of those classes of goods which are loaded and unloaded by their owners. Turning, therefore, to the report of the S. B. & N. Y. R. R., we see that it carried 533,355 tons, of which 28,126 tons were reported as "of the products of the forest," presumably lumber ; and 442,764 tons of "other articles," presumably mostly coal. This leaves but 62,465 tons to be handled, while the amount charged to "loading and unloading," amounts to $14,362.19, or 23.4 cents per ton of miscellaneous freight. On the Harlem R. R., on the other hand, there are but 170,779 tons reported as of "products of the forest" and "other articles," out of 377,537 tons transported, so that it is presumed there were 206,758 tons to be handled, at a cost of $41,817.55 for "loading and unloading," or 20.2 cents per ton. A similar calculation for the Erie Railway gives a cost of 25.8 cents per ton handled ; so that the road which actually did its work the cheapest, finds this expense the greater burden, in consequence of the peculiarities of its traffic.

We come here upon an important cause of difference of cost of transportation between different lines. It is found not only that the cost of loading and unloading, checking and billing goods, is independent of the number of continuous miles which they are transported over the line, but also that this arbitrary charge varies greatly upon different roads, in consequence of the nature of their business, and the

proportion of it which requires handling. This may go far to explain the general prosperity of the coal roads, which, while they have been enabled to obtain very nearly the same rates as other lines, have been put to far less expense in the handling of their tonnage and management of their business.

While, therefore, the character of the traffic on each road compels a certain expense for station service, the length to which it is to be hauled, governs the resulting cost per ton per mile. To illustrate the manner in which this element alone varies the cost of the traffic, the following theoretical table has been constructed on the basis of the cost for 1872, on the New York Central, and the supposition that while the cost of station service remains the same per ton, all the other expenses vary in proportion to the train miles.

TABLE.

Showing the effect of arbitrary charges for station expenses upon the average train of 130 tons. The station expenses being assumed at 31.62 cents per ton handled, and all the remaining expenses at $1.1704 per mile run by train:

$$10 \text{ miles cost } \frac{130 \times 0.3162 + \$1.1704 \times 10}{130 \times 10} = 4.062 \text{ cts. ₱ ton ₱ mile.}$$

$$20 \text{ " " } \frac{130 \times 0.3162 + \$1.1704 \times 20}{130 \times 20} = 2.481 \text{ " "}$$

$$50 \text{ " " } \frac{130 \times 0.3162 + \$1.1704 \times 50}{130 \times 50} = 1.533 \text{ " "}$$

$$100 \text{ " " } \frac{130 \times 0.3162 + \$1.1704 \times 100}{130 \times 100} = 1.216 \text{ " "}$$

$$200 \text{ " " } \frac{130 \times 0.3162 + \$1.1704 \times 200}{130 \times 200} = 1.058 \text{ " "}$$

$$232 \text{ " " } \frac{130 \times 0.3162 + \$1.1704 \times 232}{130 \times 232} = 1.037 \text{ " "}$$

$$500 \text{ " " } \frac{130 \times 0.3162 + \$1.1704 \times 500}{130 \times 500} = 0.963 \text{ " "}$$

$$1{,}000 \text{ " " } \frac{130 \times 0.3162 + \$1.1704 \times 1{,}000}{130 \times 1{,}000} = 0.932 \text{ " "}$$

The above, however, is for the average train of 130 tons, which is thus small in consequence of the short runs consequent upon local business. If beyond 200 miles, therefore, we assume that the trains consist of 24 loaded cars, or 240 tons, the cost becomes:

$$200 \text{ miles cost } \frac{240 \times 0.3162 + \$1.1704 \times 200}{240 \times 200} = 0.645 \text{ cts. ₱ ton ₱ mile.}$$

$$500 \text{ miles cost } \frac{240 \times 0.3162 + \$1.1704 \times 500}{240 \times 500} = 0.551 \text{ cts. ℔ ton ℔ mile.}$$

$$1{,}000 \text{ " " } \frac{240 \times 0.3162 + \$1.1704 \times 1{,}000}{240 \times 1{,}000} = 0.519 \text{ " "}$$

If, however, upon arrival at their destination, there is not enough return tonnage to load more than 10 of these cars, and the remaining 14 must be hauled back empty, the cost of the return tonnage is as follows :

$$200 \text{ miles cost } \frac{100 \times 0.3162 + \$1.1704 \times 200}{100 \times 200} = 1.328 \text{ cts. ℔ ton ℔ mile.}$$

$$500 \text{ " " } \frac{100 \times 0.3162 + \$1.1704 \times 500}{100 \times 500} = 1.255 \text{ " "}$$

$$1{,}000 \text{ " " } \frac{100 \times 0.3162 + \$1.1704 \times 1{,}000}{100 \times 1{,}000} = 1{,}202 \text{ " "}$$

It has here been assumed, that all the expenses, except station service, vary in proportion to the miles run by freight trains. This is in excess of the truth. We have seen that "roadway charges" and "general expense" cannot so vary, being mainly controlled by other circumstances ; and if we now turn to the column of track repairs in the table, we find that it varies in cost from 20 to 39 cents per mile run by trains, and a further inspection shows considerable variations in cost between the different roads, under all four of the standards adopted, and indicates that the cost does not vary in direct ratio to the business done. Thus, while the Syracuse, Binghamton & New York R. R. has expended $881.79 per mile operated for track repairs, the resulting charge is 0.19 of a cent a ton a mile ; and the Rome, Watertown & Ogdensburgh, which has spent but $582.35 per mile operated, nevertheless finds the charge 0.57 of a cent a ton a mile. In general terms, the three roads of lightest traffic in the table, find the cost of track repairs per ton per mile a burden about three times as great as the other lines.

The wear upon the track is produced by three elements ; 1st, the locomotive ; 2d, the cars ; and 3d, their contents. It is, moreover, affected by the speed, and the gradients and curves. It cannot, therefore, be expected to be in any constant relation with the contents of the cars, or the tonnage of the road, except so far as this dictates the character of the trains which must be run, the proportion of empty cars which must be hauled to provide for the business, and the full or partial loads for the cars, or for the locomotives, as a through or a local traffic predominates.

It is believed by the writer that a very large part of the wear and destruction of iron rails is due to the action of the locomotives. The

exigencies of the service having led to the modern practice of placing upon driving wheels weights which approximate closely to the crushing point of iron, the locomotive alone, probably, does more mischief to the rails than the rest of the train. If this be true, it would follow that roads with a heavy traffic cannot afford to lay any rails but steel in their tracks, while it is probable that many lines now laid with iron, whose business is yet to be built up, cannot afford to run the heavy locomotives which most of them are provided with, and that they would save large future renewals by exchanging them for lighter engines. It will be noticed that track repairs alone amount to about 20 per cent. of the cost of transporting freight, so that the importance of selecting the very best material for the track becomes at once apparent. It is chiefly from this, in connection with improved methods of car and train service, that future cheapening of transportation is to be expected.

The unsatisfactory result of the analysis of "track repairs," contained in the table, may result from the fact, that as renewals of rails are made at periodical intervals, some of the roads may have expended more or less than just enough to make good the annual wear. In order, therefore, to obtain results of value, the comparison should be extended over a series of years. For the present, however, we may consider the analysis as indicating that the charge imposed by track repairs is not in direct ratio, either with the miles operated nor with the train miles, nor yet with the tons transported one mile. It will also vary over different parts of the same road in consequence of curves, grades, variety of soil or character of traffic, so that it is not probable that it has the value of a constant charge of cost per ton per mile, either upon different roads or even upon different parts of the same road.

A comparison of the cost of "car service," per mile run by freight trains, shows a range from 7 cents to 24 cents per mile run. This may partly be caused by the difference in gradients upon the road, or the character of their locomotives, thus limiting the maximum train which may be hauled. We see, however, that while the cost was 19.63 cents per train mile on the New York & Harlem R. R., and this imposed a charge of 0.37 of a cent a ton a mile on the traffic, on the New York Central, where it was 17.60 cents per train mile, it imposed a charge of but 0.14 of a cent per ton per mile. The table shows considerably lower cost on roads doing a through, than on those wholly confined to a local traffic. This evidences the value of the component of time in this item of the cost; a car frequently requiring as much time to go to a station

10 miles distant, to be unloaded, reloaded if possible, and returned, as to carry the same load 300 miles ; or that, as stated in the Report of the Massachusetts Railroad Commissioners for 1872, "wheels earn money only while they are in motion."

The cost of car service is also dependent upon the proportion of empty cars which the service requires to be hauled. It has been shown, while treating of station service, that if a return load can be obtained for only 10 cars out of 24 (a not unusual proportion on most of the roads tabulated), the cost per ton per mile of their contents will be more than doubled. This again indicates the smaller cost of through, as compared with local business, it being far more easy to obtain a return load promptly from a terminal than from a local station. Indeed, it is probable that were a stream of through traffic, say of one million tons a year, thrown upon the Harlem road, the cost per ton per mile on that line, which is now 3.63 cents, would at once be reduced to about one-half that amount, so that it might cheapen its charges to all its patrons.

It is to be regretted that the reports to the State Engineer do not give the mileage made by freight cars, as this would furnish an excellent basis of comparison. It might, perhaps, be obtained by multiplying the train miles by the return giving the average weight of freight trains, were the latter correct ; but they have been so evidently guessed at, as to possess no value.

When we take up the "train service"—the transportation proper, we expect to find at least some uniformity of cost between the different roads, some regular element of expense, and some solid ground on which to base a claim, that their charges shall be uniform. We refer to the table, and we find instead, that the cost varies per ton per mile from 0.203 of a cent on the Syracuse, Binghamton & New York R. R., to 1.18 cents on the Harlem R. R.; or if we examine it by train miles, that it costs from 25 cents to 62 cents to run a train a mile. In order to account for this, the table on the next page has been made up of the cost, in detail, of running a freight train a mile upon the different roads for the year ending September 30th, 1872.

We see that while the repairs of engines have been pretty uniform, there are large differences in the cost of train hands, and especially in that of fuel. On the Central it is twice as great, and on the Rensselaer & Saratoga nearly three times as great, as upon the Erie, in consequence of their respective distances from the coal fields. The difference in cost of train men is to be attributed partly to variations in wages, but more

particularly to the peculiarities of the road and traffic, which require a certain number of men to manage the trains, or to assist in handling the local freights at way stations. The same differences will be found to exist, though in a less degree, perhaps, over parts of the same road, especially if some portions are worked as branches and some as main line, and it would seem to follow that if strict regard to cost were had in adjusting the charges, they would have to be different for similar distances on the same road and its branches, in accordance with the character of the gradients, peculiarities of traffic, distance from fuel supply, &c., &c.

COST OF FREIGHT TRAIN SERVICE PER MILE.

	New York Central.	Erie.	Lake Shore & Michigan Southern.	Syracuse, Binghamton & New York.	New York & Harlem.	Rensselaer & Saratoga.	Rome, Watertown, and Ogdensburg.
	$	$	$	$	$	$	$
Conductors and brakemen......	0.0540	0.0931	0.0688	0.0251	0.2187*	0.0316	0.0780
Enginemen and firemen........	0.0850	0.0889	0.0795	0.0318	0.1093	0.0832	0.1078
Fuel........................	0 1822	0.0873	0.1487	0.1103	0.1524	0.2363	0.1983
Oil and waste.................	0.0167	0.0131	0.0146	0.0130	0.0222	0.0166	0.0098
Repairs of engines..............	0.0946	0.0797	0.0832	0.0623	0.1128	0.0837	0.1000
Repairs of tools (half of total)...	0.0045	0.0045	....	0.0027	0.0039	....	0.0055
Water........................	0.0091	0.0037	....	....	....	0.0024	0.0042
Incidentals....................	0.0661	0.0028	0.0030	0.0084	0.0044	....	....
	0.5122	0.3731	0.3978	0.2536	0.6237	0.4538	0.5036

In every case, moreover, it will be noticed that the train service amounts to but about one-third of the total expense per ton per mile. Taken in connection with the car service, it is scarcely one-half of the whole, and the cost of these two elements varies from 33 cents to 82 cents per train mile, the roads of largest business varying from 53 cents to 69 cents; while the total cost of operating is from 88 cents to $1.92 per mile run by freight trains. Thus the transportation proper, which almost every one has in mind, when discussing railroad charges, costs but about one-half of the cost of the total service rendered to the public. The other expenses are in some degree fixed or arbitrary charges, or they do not vary with the distance to which the goods are conveyed, and burden the business more or less in proportion to its volume or character.

* This item is so greatly in excess of every other, as clearly to indicate a discrepancy in the method of keeping the accounts.

As to the "insurance," or losses incurred on goods in transit, it seems surprisingly small. It scarcely amounts to two per cent. of the whole expenses, or probably to about one-tenth of one per cent. upon the value of the articles carried. Perhaps a comparison for other years would make a less favorable showing than that selected as the basis of the table.

One important element has remained thus far entirely unnoticed, and nothing has been said about the interest upon the capital invested in the road and its equipment. This is as legitimate a charge upon the traffic as the cost of running the trains, and it must be covered by the profit charged upon the various shipments. In apportioning these profits, it becomes necessary, not only to consider the varying cost of each particular class of shipment, its volume, the expenses it occasions, the bulk or space it occupies in the cars, and the risk incurred from its perishable properties, but also to use sound judgment as to its comparative value, and the amount of profit it will bear, so as to adjust the burden of transportation where it will least be felt.

It is well understood by railroad managers, that the maximum of aggregate profit is by no means coincident with high charges. Every reduction of rates brings out for transportation more and more of the bulky and cheap commodities, and permits their shipment at a profit. This again cheapens the average cost of the whole by spreading the fixed charges, and those which do not increase with the traffic, over a greater number of tons; but as, in the meantime, the railroads must pay their operating expenses, and, if possible, interest on their cost, and as the business is not capable of indefinite extension, the adjustment of rates becomes a delicate operation, which requires careful experimenting in order to ascertain the rates which at any given time will yield a maximum of profit.

The universal tendency of rates in this country has hitherto steadily been downward. They sometimes have had to be raised on particular articles, but it has so invariably been found that those lines proved most profitable which developed the largest tonnage by adjusting the rates, so as to admit of the shipment of cheap articles, that it is deemed good policy to make the charges just as low as experiment proves to be prudent. The owners of new railroads, therefore, have generally been content to wait some years for full returns upon their investments, in order to promote the development of the country and of a large tonnage. But as a compensation they have collected more than the legal interest, whenever the growth of traffic has enabled them to obtain it.

It thus appears from this imperfect analysis of the table, which it must be remembered, merely contains the average results of the very numerous and dissimilar transactions carried on upon each road during the year, that railroads do very much more than merely to transport the traffic which is entrusted to them. They furnish the road and rolling stock, and keep them in repair, and they load, unload and insure the goods, in addition to carrying them. Some of these expenses have to be incurred whether a large or small business is done; many are independent of the distance which the article is transported, and others again are regulated by the character of the traffic. The cost varies with the season, with the character of the business, the value of the goods, and with the volume of tonnage which the year's crop, or prices in distant markets, brings forward for shipment, and it not unfrequently happens that a railroad knowingly transports a crop out of its tributary country at a loss, in order to furnish its patrons with the means of purchasing their annual supplies, and thus furnish profitable return tonnage for the line.

It will be seen that there is no term of comparison more fallacious, to apply to individual cases or particular shipments, than the average cost per ton per mile. This cost not only varies between the different roads, but it also varies greatly on the same road, either for different distances, or for the same distance over different parts of the line and branches. It varies with the class of goods, which requires more or less handling, and it varies even on the same goods, between the same stations, if empty cars are required to be hauled in one direction or the other. The transportation proper, including car service is but about one-half of the whole expense, and even this varies with the cost of wages and of fuel. Railroad managers, therefore, in fixing rates to be charged, must estimate and weigh, as well as they can, the average cost of a great many different operations and contingencies, and make many attempts before they can ascertain the exact rates which will give the most profitable volume of trade, and the best returns upon the investment.

It would seem to follow therefore, that the claim which is sometimes made, that charges for short distances shall be in proportion to those for long distances, is not founded on justice, and that tariffs cannot be made uniform, either upon different roads, nor for like distances on the main stem and branches of the same road, under dissimilar circumstances of traffic. Neither can the rates be made permanent as to time, for although, doubtless, they should be changed far less frequently than they are, the cost is dependent upon so many varying elements, that a fair rate one year may be either insufficient or extortionate the next.

Even the claim, much better founded, that a higher rate shall not be charged for an intermediate than for a through distance, may be unjust in practice; the extra handling required, the furnishing of empty cars, or the time lost by demurrage, may make the cost greater for the short than for the long distance. It should be stated, however, that it is believed to be good economy, as well as sound policy, not to make a higher charge from an intermediate than from a terminal station, and that the most ample notice should be given of proposed changes in the tariff, which should be uniform in its application. The cost in the majority of cases is no greater, and the ill feeling which is sure to be engendered by the contrary practice, more than offsets any resulting profits.

It follows, also, that until experience and discussion shall enable us to understand the subject better, the making of a freight tariff is, and must be to a great extent, a tentative and experimental process, which cannot as yet be governed by any fixed mathematical rules. When, therefore, a freight tariff has been adopted upon a road, no matter upon what theory of the probable cost it has been based, it is straightway found necessary to modify it, either in part or by giving special rates, in order to conform with the particular circumstances of the case. As various communities prosper or retrograde, as certain articles increase or diminish in supply or in demand, changes have to be made in the rates in order to adjust them to the new relative importance or cost of the business.

To one unacquainted with the subject, these changes in the tariff seem, and some in fact are, arbitrary and unjust. It is perhaps from this very process of adjusting rates to cover a variable cost, which it must be admitted has not always been wisely or honestly done, that the great dissatisfaction at present existing with the railroads in the West originated a dissatisfaction which has led to legislation likely to be as tentative and experimental as the tariffs it seeks to control. In spite of this legislation, the result of this analysis of the cost upon the New York roads, renders it not improbable that the tendency will be in the future towards even greater diversity in charges than now exists upon roads in different parts of the West. Some of them are not now earning enough to meet their operating expenses, to say nothing of future renewals or returns upon the investment, and if they are ever to become profitable, they will be led to raise their freight rates if they can. This, however, does not wholly depend on the will of the managers. It is regulated by competition, and by the charges which the goods to be moved can afford to pay; so that the result of advancing the rates, upon many roads which have

been built, would be to drive away or destroy the small amount of business which they now enjoy.

A country is enabled to sustain a railroad, pretty much in the ratio of the tonnage of its annual exportable products, and this determines, therefore, the distances apart at which it is most profitable to build them. This, in an agricultural section, varies with the crops which experiment shows it most profitable to raise. Thus in the cotton States, a single railway car carries off the product of 44 acres, assuming an average of half a bale per acre, and the roads are built about 60 miles apart. In Illinois, the proximity of the lakes makes it profitable to ship corn, which only requires the product of 7½ acres, at 50 bushels to the acre, to load a car, and the roads have been about 20 miles apart. In a region growing wheat, a car would carry off the annual product of 22 acres at 15 bushels to the acre, while in a pastoral country, it would require the product of 40 acres in cattle, or of 50 acres in hogs to furnish a single carload. As population proceeds westward, therefore, and engages in the raising of those products of greater concentrated value, in proportion to their bulk, which alone will bear the cost of transportation to a distant market, it must expect in the long run to submit to higher rates of rail transportation in the proportion of the diminished tonnage it will be enabled to furnish.

The same effect is produced by the building of more roads than are required to do the business of a portion of the country, under the mistaken belief that competition cheapens rates under all circumstances. The new roads, if in excess of the demand, while unprofitable themselves, will raise the cost upon the older lines, by diminishing their tonnage, and the consequent base upon which their fixed charges are to be apportioned. It is quite possible, therefore, that the recent extensive building of new roads in the West, while greatly adding to the convenience of the public, and cheapening transportation from the farm to the nearest railroad station, may yet result in an increase of freight charges, whenever the class of goods to be conveyed will bear any additional burden.

It is, perhaps, the indistinct recognition of these facts which led to the system of granting subsidies, either in lands, municipal bonds or money, under which so many new roads have recently been constructed. This system has probably now passed the period of its usefulness, having induced the building of almost all roads really needed by the country, as well as a good many besides, which will not be profitable for many years, and which the owners are now very sorry to have built. Its true

theory seems to be that the subsidy should form a fund, to defray the interest upon the capital invested in the road, during the years necessary for the development of its traffic, without interfering with the growth of the country, by imposing high rates of transportation. It is greatly to be regretted that it has been perverted in some cases, either by applying the subsidies towards the construction of the road, or by dividing them among the managers, instead of setting them apart as a reserve fund, to tide over the first eight or ten years of unprofitable business.

It would lead us much too far here to discuss even the approximate cost of any particular class of shipments, or the fair rate which should be charged for it. This may roughly be done, by the general method here adopted while discussing the station service, of considering the cost of handling, billing and checking the freight, as an arbitrary charge per ton, irrespective of distance carried, varying with the character of the goods; and apportioning the other expenses in proportion to the miles run by freight trains, restoring, however, to each element its true value for the particular case in hand. This can only be done by taking up in great detail the accounts of a single road, upon which the particular shipment is to occur, and the result will chiefly be valuable for that road, and not for others; for even after the cost is ascertained, upon an assumed volume of business, it becomes a matter of judgment as to how much profit shall be charged in order to secure the maximum of revenue. All the previous calculations may thus be modified by the resulting and possibly different volume of traffic, consequent upon the introduction of the element of profit; while the subject more properly belongs to a discussion upon the relations of the railroads to the public.

The object of the remarks which led to the writing of the present paper, was to show how and why the cost and charges differed upon various roads. That purpose will be accomplished, if what has been said tends to promote such investigation and discussion by others, as to lead to a better understanding of the subject, and to the further investigation of the principles which govern the cost of railroad traffic.

TABLE I.

AVERAGE FREIGHT CHARGES AND COST PER TON PER MILE ON RAILWAYS IN NEW Y

Year.	New York Central.			Erie.			Lake Shore & Michigan Southern.			Syracuse, Binghamton & New York.		
	Average Charge.	Average Cost.	Road Operated.	Average Charge.	Average Cost.	Road Operated.	Average Charge.	Average Cost.	Road Operated.	Average Charge.	Average Cost.	Road
	Cents.	Cents.	%	Cents.	Cents.	%	Cents.	Cents.	%	Cents.	Cents.	
1863	2.38	1.55	62.79	2.09	0.95	61.51	2.78	1.40	62.43	1.37	0.55	4
1864	2.72	2.00	71.91	2.33	1.45	66.27	3.29	2.10	60.45	1.29	0.55	4
1865	3.31	2.52	77.87	2.76	1.99	70.69	3.29	3.13	69.67	1.49	1.57	8
1866	2.92	2.07	75.49	2.43	1.65	72.58	3.45	2.71	67.30	1.48	0.92	6
1867	2.53	1.89	76.20	2.04	1.47	72.18	3.41	2.87	77.38	1.59	0.88	5
1868	2.59	1.64	64.24	1.92	1.35	77.51	3.46	2.63	78.07	1.53	0.89	6
1869	2.21	1.30	58.09	1.60	1.17	79.29	2.83	1.58	68.73	1.92	1.50	7
1870	1.86	1.15	63.36	1.38	0.98	74.62	1.59	1.04	62.93	1.42	0.76	5
1871	1.65	1.01	61.80	1.43	1.01	71.06	1.46	0.97	66.70	1.61	1.24	7
1872	1.59	1.04	64.29	1.53	1.04	68.55	1.34	0.95	69.99	1.42	0.71	5

TABLE II.

ANALYSIS OF THE COST OF TRANSPORTING FREIGHT ON VARIOUS RAILRO

	New York Central.	Erie.	Lake Shore & Michigan Southern.
Total freight earnings	$17,258,768 20	$14,737,251 50	$12,030,409 00
Total freight expenses	$10,648,937 14	$9,859,207 80	$8,397,736 73
Total miles operated	1,310.2	1,200.5	1,198.
Average through tons‡	779,201	791,927	740,278
Average distance carried	232.34	170.86	209.10
	COST PER TON PER		
	cents.	cents.	cents.
Roadway	0.107	0.099	0.113
General expense	0.023	0.039	0.028
Station service	0.136	0.187	0.158
Track repairs	0.231	0.191	0.204
Car service	0.137	0.145	0.130
Train service	0.397	0.353	0.303
Insurance	0.012	0.023	0.011
Totals	1.043	1.037	0.947
Total ton miles	1,020,908,885	950,708,902	886,853,169

* Additional freight expenses……
† Additional cost per ton per mile.. } on Harlem for city transportation by horses……

‡ Ton miles divided by miles of road reduced to equivalent single track.

E I.

NEW YORK FOR TEN YEARS, AS STATED IN STATE ENGINEER'S REPORT.

Binghamton York.		New York & Harlem			Rensselaer & Saratoga.			Rome, Watertown & Odensburgh.		
Cost.	Road Operated.	Average Charge.	Average Cost.	Road Operated.	Average Charge.	Average Cost.	Road Operated.	Average Charge.	Average Cost.	Road Operated.
nts.	%	Cents.	Cents.	%	Cents.	Cents.	%	Cents.	Cents.	%
55	42.89	3.38	3.27	60.72	5.25	3.52	65.00	2.92	2.14	52.37
55	41.55	5.55	4.90	75.78	6.85	3.95	49.59	3.28	2.67	55.32
57	89.88	6.37	5.68	74.69	6.89	5.55	64.60	4.13	4.16	69.17
92	65.85	5.88	4.10	59.70	7.28	5.49	70.81	3.68	3.43	65.83
88	56.02	7.23	4.87	56.60	6.90	5.27	72.80	3.74	2.99	55.72
69	62.16	7.62	5.78	64.31	....	....	....	3.58	3.24	74.82
50	74.47	7.32	4.88	65.44	2.81	2.06	67.67	3.70	2.84	62.35
76	58.34	6.57	4.45	62.21	3.95	2.65	59.63	3.75	2.92	64.12
24	75.83	6.91	4.92	60.64	3.74	2.98	72.02	3.82	3.19	68.19
71	54.68	6.14	4.34	60.19	3.27	2.31	65.45	2.93	2.64	75.49

II.

RAILROADS FOR THE YEAR ENDING SEPTEMBER 30TH, 1872.

e & n .	Syracuse, Binghamton & New York.	New York & Harlem.	Rensselaer & Saratoga.	Rome, Watertown & Ogdensburg.
0 00	$556,091 03	$1,560,376 80	$1,013,853 09	$702,769 67
5 73	$265,172 64	$764,844 10*	$679,228 07	$592,895 56
98.	81.	168.77	181.	222.
,278	464,689	125,405	162,717	101,134
9.10	70.57	55.73	52.88	66.73
PER MILE.				
	cents.	cents.	cents.	cents.
.113	0.172	0.614	0.334	0.408
.028	0.008	0.126	0.109	0.125
.158	0.062	0.624	0.320	0.365
.204	0.190	0.687	0.635	0.576
.130	0.058	0.371	0.158	0.364
.303	0.208	1.180	0.735	0.768
.011	0.011	0.033	0.015	0.035
.947	0.704	3.635†	2.306	2.641
,169	37,639,618	21,039,166	29,451,790	22,451,836

.... { $148,919 23 cents 0.708 } making total.. { $913,763 33 cents 4.343

TABLE II.—(*Conti*

ANALYSIS OF THE COST OF TRANSPORTING FREIGHT ON VARIOUS RAILRO

COST PER MILE OPE

	New York Central.	Erie.	Lake Shore & Michigan Southern.
Roadway	$831 20	$786 65	$835 93
General expense	182 08	306 43	206 21
Station service	1,060 38	1,477 73	1,170 53
Track repairs	1,802 15	1,516 67	1,508 54
Car service	1,062 97	1,143 51	964 94
Train service	3,093 00	2,798 19	2,241 76
Insurance	93 94	183 40	81 89
Totals	$8,127 72	$8,212 58	$7,009 80
	COST PER MILE RUN BY FR		
Roadway	$0 13.77	$010.50	$0 14.83
General expense	0 03.01	0 04.08	0 03.66
Station service	0 17.56	0 19.71	0 20.77
Track repairs	0 29.85	0 20.22	0 26.76
Car service	0 17.60	0 15.25	0 17.12
Train service	0 51.22	0 37.31	0 39.78
Insurance	0 01.59	0 02.44	0 01.45
Totals	$1 34.60	$1 09.51	$1 24.37
Total freight train miles	7,911,257	9,004,051	6,752,291
	COST PER TON TRANS		
Roadway	$0 24.78	$0 16.97	$0 23.71
General expense	0 05.43	0 06.61	0 05.85
Station service	0 31.62	0 31.88	0 33.20
Track repairs	0 53.74	0 32.72	0 42.79
Car service	0 31.69	0 24.67	0 27.37
Train service	0 92.23	0 60.37	0 63.59
Insurance	0 02.86	0 03.96	0 02.32
Total	$2 42.35	$1 77.18	$1 98.84
Total tons carried	4,393,965	5,564,274	4,223,434

* Additional cost per ton transported on Harlem, for city transportation by horses......$

Continued.)

RAILROADS FOR THE YEAR ENDING SEPTEMBER 30TH, 1872.

OPERATED.

... & ... n	Syracuse, Binghamton & New York.	New York & Harlem.	Rensselaer & Saratoga.	Rome, Watertown & Ogdensburg.
93	$802 21	$765 23	$544 93	$412 58
21	39 40	157 42	176 64	126 33
53	286 42	778 57	520 17	369 12
54	881 79	856 01	1,034 22	582 35
94	271 10	462 87	256 50	367 77
76	943 07	1,470 87	1,196 29	777 29
89	49 74	40 91	23 89	35 26
80	$3,273 74	$4,531 88	$3,752 64	$2,670 70
Y FREIGHT TRAINS.				
.83	$0 21.57	$0 32.45	$0 20.67	$0 26.73
.66	0 01.06	0 06.68	0 06.70	0 08.18
.77	0 07.70	0 33.02	0 19.73	0 23.91
.76	0 23.72	0 36.30	0 39.23	0 37.73
.12	0 07.29	0 19.63	0 09.73	0 23.83
.78	0 25.36	0 62.37	0 45.38	0 50.36
.45	0 01.34	0 01.74	0 00.91	0 02.29
.37	$0 88.04	$1 92.18	$1 42.35	$1 73.03
291	301,200	397,985	477,157	342,654
RANSPORTED.				
.71	$0 12.18	$0 34.21	$0 17.70	$0 27.23
.85	0 00.66	0 07.04	0 05.74	0 08.34
.20	0 04.35	0 34.80	0 16.94	0 24.36
.79	0 13.39	0 38.27	0 33.60	0 38.42
.37	0 04.12	0 20.69	0 08.34	0 24.27
.59	0 14.32	0 65.75	0 38.87	0 51.29
.32	0 00.76	0 01.83	0 00.77	0 02.33
.84	$0 49.72	$2 02.59*	$1 21.96	$1 76.24
,434	533,355	377,537	556,931	336,440

s......$0 39.45......making total......$2 42.04.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INCORPORATED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

LXXIX.

THE APPROXIMATE VALUE OF A REDUCTION OF RULING OR MAXIMUM GRADES,

ESPECIALLY FOR THE USE OF ENGINEERS ON LOCATION OF RAILROADS.

A Paper by JOHN G. CLARKE, C. E., Member of the Society,

READ MARCH 18TH, 1874.

At the request of some professional friends, the following method of obtaining approximately the value of the reduction of a ruling, or maximum grade on a railroad, is respectfully submitted for the consideration of the Society. For the basis of the paper, the writer is indebted to the late Charles Ellet, Jr.

The want long felt by engineers on location, of some method by which they may feel assured that the expenditure which they make to reduce the ruling grade is legitimate and proper, has given birth to many empyrical formulas, none of which, so far as known, are at all satisfactory. It is not claimed that the formula now presented is exact; because the very foundation—the amount of business that the road shall do—is conjectural, and in order to obtain a formula which can be applied readily, values which are not mathematically accurate are admitted, but these errors are of small importance, and it is thought that when a pretty close estimate of the value of the grade, upon the conjecture that it is to do a certain amount of business, is arrived at, it will be near enough for all practical purposes.

The value of the reduction being the effect which such reduction will have on the cost of transportation, this cost is divided under three heads:

First—The cost of repairs, &c., to road (not including cost of relaying track). This is evidently *constant per mile.*

Second—Deterioration, caused by transportation of materials, as repairs to track due to weight of freight, expense of repairing and renewing cars, also expense of agents, force of all kinds, and contingencies, with interest on cost of cars; the items under this head are *constant per ton per mile.*

Third—Locomotive power; this embraces expense of repairs to track due to weight of engine and tender, cost of repairing and renewing engine and tender, interest on first cost, cost of fuel, &c., and wages of engineman, fireman and conductor. This item of cost is *very nearly constant per mile run by the engine*, and for this discussion is assumed at 40 cents; the proper number for one class of engines would, of course, be slightly changed for another, but the variation from the average is not large.

In speaking of quantities as constant, it is evidently meant that they will be constant for any particular road, regardless of grade; they will, of course, vary with each road, depending upon locality. management, traffic, &c. The cost of the items under the first two heads being independent of the grade, its value is therefore to be measured by the reduction of the cost of motive power which it entails, or what is the same, the reduction of the total number of miles which the engines are obliged to travel in consequence of being able to carry the larger load; and thus, it is not the height of the summits, but the *rate* of ascent that is to be considered.

Let W represent the gross weight in tons in full trains carried annually one mile in the direction opposed to the maximum grade, and A represent the average gross load in tons an engine can draw up this grade, then $\frac{2\,W}{A}$ will equal the number of miles the engines travel annually, and $\frac{2\,W}{A} \times 0.40 = \frac{4\,W}{5\,A}$, will be the annual cost of motive power in dollars.

The rolling friction of car wheels being taken at $\frac{1}{330}$ of the weight, or about 6 pounds per ton, the force of gravity and friction on a grade of 16 feet per mile is twice the resistance of friction on a level; on a 32-foot grade, three times; on a 48-foot grade, four times, &c. Then if T equal the gross load an engine can draw on a level, and X equal the maximum grade—

$$T \times \frac{16}{16 + X} = A,$$

and as on low grades the weight of the engine is small in proportion to

the weight of the train which it draws—in order to obtain a formula which can be used—its weight may be assumed to vary with the weight of the train, and thus the formula found above may be applied to the train alone, without considering the weight of the engine.

In the very able report of Mr. Silas Seymour, on the change of grade on the Union Pacific R. R., west of Omaha, he carefully worked out a table showing the number of cars his engines would haul on different grades, in which he has even taken into consideration the effect of the wind on his train moving at a given velocity; below is a table giving Mr. Seymour's loads in cars on grades from 10 to 80 feet, and opposite, the numbers of cars given by the above equation, discarding from it the weight of the engine:

Grades.	Seymour's Report.		Formula.	
Feet.	Cars.	Differences.	Cars.	Differences.
0	94		94	
10	56	38	58	36
20	40	16	42	16
30	30½	9½	32½	9½
40	25	5½	27	5½
50	20½	4½	23	4
60	17	3½	19½	3½
70	15	2	17	2½
80	13	2	15½	2½

By this table it will be observed that the differences between Mr. Seymour's figures, which are accepted as accurate, and those given by the formula, are nearly alike; and, therefore, the calculated *values* of any modification of grade from either column will be almost identical for any grade less than 100 feet per mile.

Substitute in the expression of the annual cost of the motive power in dollars, $\frac{4}{5}\frac{W}{A}$, the value of A, just found, and it becomes $\frac{16 + X}{20\ T}\ W$. Now, if X assume a new value X^1, the difference in cost of motive power between the two grades will be

$$\frac{16 + X^1}{20\ T}\ W - \frac{16 + X}{20\ T}\ W = \frac{X^1 - X}{20\ T}\ W.$$

For application, let $X^1 - X = 1$, then $\frac{W}{20\ T}$ will be the increment in the annual working cost for each foot of the grade.

As an example—assume 940 tons, the load which Mr. Seymour takes as the gross load an engine can draw on a level on his road, then $\frac{W}{20\ T} = \frac{W}{18800}$; capitalize this (money taken at 7 per cent.), and $\frac{1}{18800} \times \frac{100}{7} = \frac{1}{1316}$ of a dollar will be the value per ton per mile for each foot taken from the ruling grade. The engine stage west of Omaha is reported to be 145 miles long, and it is presumed that this is found to be the economical run for the engine; if Mr. Seymour's figures, 112,800 tons, for the probable business to be done against the heavy grade, are taken as the gross tonnage conveyed through in the direction of the grade, the sum one would be justified in spending to reduce the grade 10 feet, will be $\frac{112,800 \times 145 \times 10}{1316} = \$124,286$.

Mr. Wilson Crosby—The line on the Union Pacific Railroad, between Omaha and the Elkhorn river, as at first adopted, showed grades of 66 feet per mile, ascending westward in two instances, and 80 feet per mile ascending eastward in three instances; and so distributed that the engine power required to work this part of the road would practically have to run over the whole distance between Omaha and the Elkhorn, 23 miles.* By this formula the value of a reduction of 10 feet in the ruling grade with the assumed movement of 112,800 tons against it annually, would be

$$\frac{112,800 \times 23 \times 10}{1,316} = \$19,714$$

instead of \$124,286, obtained by supposing the engines to all run a distance of 145 miles west of Omaha. If the grades were made to conform to 30 feet per mile rising west beyond the Elkhorn river, the amount of expenditure justifiable to accomplish this then would be

$$\frac{112,800 \times 23 \times (66-30)}{1,316} = \$70,971.$$

* West of Elkhorn the grades were level, or ascending gently westward. As the idea of reducing the grades between Omaha and the Elkkorn to 30 feet per mile was entertained, it is presumable that those in the Platte Valley did not exceed this limit, and that 30 feet per mile, rising west, may be taken as the rolling grade on all except the section between Omaha and the Elkhorn.

The annual business to be anticipated when converted into freight car-loads, according to Mr. Seymour's standard, gives 45,000 car-loads westward and 34,700 car-loads eastward (including the return of empty cars). By reference to the table it will be seen that an amount of force capable of transporting 45,000 car-loads up grades of 66 feet per mile, will be more than sufficient to haul 31,700 car-loads in the opposite direction up grades of 80 feet per mile. For this proportion of traffic 39 feet per mile ascending east would be an equivalent to 30 feet ascending west. The 66 feet rising west is the ruling grade on this part of the road as located originally.

LXXX.

AN ACCOUNT OF THE OPERATION OF THE GUNPOWDER PILE-DRIVER.

A Paper by SAMUEL R. PROBASCO, C. E., Member of the Society.

READ MARCH 5TH, 1873.

The machine known as "Shaw's patent gunpowder pile-driver" was set at work in October, 1872, on a line of sheet piles, to be driven for a dam now in process of construction for a reservoir in the valley of Parsonage creek on Long Island. The material in which the piles are driven is sand, mixed with fine gravel, in places cemented together by a deposit of iron, which renders it excessively hard, and difficult to move with the aid of a pick, and causes it to resemble the traditional "hard pan." It was found in patches over the entire area to be used for the site of the dam, and at varying depths. The existence of clay in pockets of varying size below the water-level of the basin is known, and some borings made, have shown it to be present at about 15 feet below the surface, in fine particles and thoroughly mixed with sand; this lower strata is excessively hard, or, rather, tough and tenacious when in place, and presents an obstacle even to a boring augur; the whole material is under water.

The machine resembles an ordinary pile-driver in form. A mass of iron bored out like a gun rests on the head of the pile, and remains there. A block of iron with a piston, which corresponds to the bore of the gun, and has no windage, is moved up by the explosion of powder within the gun. As soon as the piston leaves the gun, a cartridge is thrown in, and the piston descending air tight, in the bore of the gun, generates heat by compressing the air within the bore, sufficient to ignite the powder, which exploding, forces the ram up and the pile down. The area of the piston is supposed to be adjusted to the weight of the ram, which also has to be adjusted to the work to be done. The cartridges are of what is known as "soda powder," made up in cylinders of 1¼ to 1½ ounces, covered with a coating of black lead and paraffine; this coating is relied upon to keep the powder dry, to lubricate the gun

and thereby preserve the requisite tightness to prevent the escape of gas and to cause the entire force to be exerted on the base of the piston. The gun and ram are of cast-iron; the latter has attached to its underside a piston of wrought-iron of little less bore than the gun; on its lower end is a steel ring which fits the bore closely. The whole device, while first working, is very pretty to look at, and seems to be all that is required.

The actual result of the performance on this work was as follows: At first, several explosions are necessary to lubricate the gun, which leaks gas so that the ram will not go to the requisite height to move the pile, but after a few shots the piston moves up regularly with each explosion, and descending, fires the charge, which moves the pile down and itself upward When the resistance is small, as in driving in mud or any other soft material, this machine may be economical, but in the material before specified, where the resistance is uniform from the start, and to force a pile down 15 to 16 feet requires over 300 blows from cartridges costing $2\frac{1}{2}$ cents each, its economic value is *nil*. The gas from the exploded powder soon eats passages in the steel ring at the lower end of the piston, and as soon as the gas can escape, the moving power of the machine is gone, and the blow fails to move the pile. The gun gets intensely hot from the rapid discharges and the number of shots required to move a pile to place in the dense material, and enlarges in its bore from this cause, whereby the gas escapes.

When first set to work 7 piles were driven with it. By this time, not only was the expense for powder very great (each pile cost more for powder than the contractor got for piles in place), but the machine rerefused to work. On examination, it was found that the steel ring was full of scores or furrows from the action of the powder, and that the piston (of 5 inches diameter) was so bent as to be useless. It was presumed this was so injured by striking the bottom of the gun, the air cushion relied upon to prevent its reaching the bottom of the bore being lost by the scoring of the ring. The inventor was then called upon to examine the machine. He stated the cause of the undue consumption of powder was owing to the area of the piston not being enough for the weight—from 1,700 to 1,800 lbs. of the ram above it. The bending of the piston was accounted for as before stated, and the action of the powder in producing furrows in the ring was held to be due to the softness of the metal in the ring. The gun and ram were sent to Philadelphia, and after two weeks were returned, with a new piston 7

inches in diameter fitted in the ram, and the gun bored out to correspond. Work was recommenced. After driving 10 piles the machine was again laid aside; the final result being the same as in the first trials, except that the piston was not bent. The gun got so hot from the repeated discharges as to fuse the coating of the cartridges, and to fire the powder before the ram got down to its place; it thereby failed to move the pile; the ring was scored and furrowed as before, and the leakage of gas was so great that there was not sufficient force to move the ram. The work of experimenting then ceased, and the driving of the piles was commenced by the usual process, and successfully accomplished at a reduced cost.

A probable cause of the failure of the machine to do this work was owing to the permanent expansion of the gun in the diameter of the bore, from the great heat caused by the frequent discharges. In Major Wade's or Capt. Rodman's Reports to the Chief of Ordnance, U. S. A., may be found tables of the enlargement of the bores of guns from 6 inches to 15 inches in diameter, and for discharges numbering from 25 to 2,500, which show a continuous expansion and permanent set. The same result must inevitably obtain in the case of the pile-driver gun; slower, of course, on account of the less amount of heat, due to the smaller quantity of powder, but the discharges are much more frequent, and the heat has less opportunity to pass away before receiving an accession. The thinness of the wall of the pile-driver gun is more favorable to its cooling than in the case of cannon, but enlargement must ensue from the heat, and as the piston and ring cannot expand with the gun in the same ratio, but slower, on account of difference in form and exposure to heat, the windage must increase with greater or less rapidity. This cause, of itself, is sufficient to warrant the opinion that pile-driving on this principle must be a failure, when the resistance is great, and the number of blows to be made involve the consumption of a large quantity of powder.

In all, 17 piles were driven by this machine, to depths varying from 14 to 19 feet, and needing from 200 to 300 blows of $1\frac{1}{2}$ ounce cartridges. Its economy is, as yet, in no way apparent in this material; as compared with the process of driving with a ram, the same number of men is needed to do the work; an engine is necessary to put the piles on end and to lift the gun and ram while setting the pile; the first cost of the machine is much greater, and so far it has failed to do the work successfully. 11 piles have been put down $15\frac{1}{2}$ feet in 10 hours (this is a

maximum effort, 8 to 9 ordinarily form a day's work), by hammer weighing 1,800 pounds, falling from 8 to 10 feet, and at a cost per pile no greater than 100 blows from the powder machine. These 100 blows, in the best working of this machine, would only put the pile down 10 feet, leaving the last 5½ feet to take an equal number, if not more, to move it to place.

The work done by the "Gunpowder pile driver" was as follows:

	Length.	Distance Driven	Number of Blows.	Cartridges to each blow.
Oct. 1st.........	20 ft. 3 in.	19 ft.	296	2 to each blow.
"	20 ft. 4 in.	14 ft.	198	No record.
Oct. 2d.........	20 ft. 5 in.	15 ft.	210	"
"	20 ft. 6 in.	15 ft.	200	"
"	20 ft.	....	117	3 to each blow.
"	20 ft.	15 ft. 4 in.	358	1 " "
Nov. 13th.......	20 ft.	15 ft.	189	No record.
"	20 ft.	15 ft. 4 in.	181	"
"	20 ft.	15 ft. 4 in.	146	"
"	20 ft.	15 ft. 4 in.	195	"
"	20 ft.	15 ft. 3 in.	133	"
"	20 ft.	15 ft. 4 in.	138	"
"	20 ft.	15 ft. 4 in.	213	"
"	20 ft.	15 ft. 4 in.	207	"
Nov. 15th.......	20 ft.	15 ft. 4 in.	172	"
"	20 ft.	15 ft. 4 in.	240	1 to each blow.

This record of the driving of each pile is complete, except that the number of cartridges used per blow is not given in every case. The cost of powder burned, excluding freight and cartage, was $233.89, an average per pile driven of $13.76, if the number (17) on which the cartridges were expended is taken in account; or of $14.61, if the number (16) for which the contractor received pay is taken.

It was noticed during the use of this machine that, judging by the distance moved, two cartridges did the best work; one was not quite sufficient, and three were too many, the blow being so hard that the pile could not move rapidly enough, and therefore crippled under the band at once. Long blows from the common machine had the same effect, while a reduction of the height from which the hammer fell lessened the tendency

of the pile to crush, and it moved down more effectually. The whole number of piles driven was 2,700, all but 16 of which were put in place by the common machine.

The piling was of spruce, 20 feet long, and varying in size from 10 x 10 inches to 10 x 14 inches; the piles were, by a machine, grooved on each side 2 x 2 inches. On one side a yellow pine strip, 4 x 2 inches, was placed in the groove and pinned every 3 feet by iron pins ½-inch in diameter and 8 inches long. The piles at the points were beveled on three sides, leaving the grooved side untouched. The groove was driven on the tongue of the preceding pile; a band was found necessary for the heads of the piles. The smaller and thinner, within limits, the bands were made, the better they stood the shock; very heavy bands failed the soonest.

The first 7 piles were driven without shoeing; the eighth pile split in the groove, and on being pulled out, the appearance of the point showed the necessity of protection. A wrought-iron shoe was impracticable, on account of cost and the difficulty of being made with a groove. Cast-iron was decided upon, and cup shoes weighing 39 pounds each, having three beveled and one plain side, and with a groove, were made. First, they were provided with flat points; it was then thought that the resistance would be lessened by making a sharp wedge on the bottom. A tendency to drift sideways, in consequence of this increased sharpness, developed at once, and no perceptible diminution of resistance was found; the piles drove very wildly, in hard ground it was almost impossible to keep them in place, and when once started wrong, no way of getting them straight again was discovered.

The sharp edge of the groove cut the tongue, and the tendency of the latter to rise with each blow was, at first, difficult to overcome. As an experiment, the tongue was cut away under the band, and a packing put in of wood, with fibres at right angles to the pile. This cushion did very well, but was troublesome to make and keep in place; a chain was then put around tongue and pile, and a long lever put in for a twist; a man with a tail rope, exerting his force to bind the tongue and pile together at the moment of the ram striking, prevented the upward movement of the tongue.

The depth to which the pile would go was uncertain, and no data being at hand, the orders from the first were to drive as far as possible. 75 piles were driven in this way, not stopping until the pile showed weakness, or gave out at once. These depths below the bottom of the reservoir were plotted to scale, and an inspection showed the mean depth attained to be

about 15½ feet. This plane was adopted for the bottom level. Since this, experience has enabled 6 inches more depth to be attained, or a depth of 16 feet, which, we find, is about the maximum penetration that can be attained in this kind of material, and this can only be accomplished with the very best dry spruce, and perfectly sound; green spruce will not bear the pounding, and any shakes or cracks are developed in a few moments by the impact of the ram.

MR. WILLIAM J. HOLROYDE*—The "Gunpowder pile-driver" is a quite simple machine. Its principles of operation and construction have been explicitly described heretofore, so that they must be familiar to the members. Since the original machine was built, improvements have been introduced from time to time, as experience suggested; wrought-iron, cast-steel and cast-iron, combined with wood, have been substituted for the old cast-iron guides, securing increased rigidity and strength, without additional weight; improvements have been made in the style of brake, and the mechanical arrangement of the whole has been substantially improved and rendered far more effective than were the cast-iron machines of a year ago. Each machine built has embodied some real advantage over its predecessor; but no deviation from the original principle has been attempted, or deemed advisable.

I have had charge of work done with steam drivers, as well as with the gunpower machines, and during the last 18 months have superintended the driving of several thousand piles by the latter method. Of these, I refer to the following:

1. For the Pennsylvania R. R. Co. at Greenwich Point, Philadelphia, for a tramway to carry locomotives and freight cars, piles averaged 10 inches diameter, placed 5 and 6 feet apart, and were driven from 15 to 20 feet into meadow not very hard ordinarily; but during the most severe weather of the winter when the work was done, the ground was frozen solid to a depth of 2 feet. The piles were neither pointed nor banded. The surface of the ground was rendered very uneven by ditches 8 feet deep and 20 feet wide, and embankments 9 feet high; the moving of the machine was therefore necessarily tedious, and rapid working impossible; cast-iron and wooden guides 40 feet high were used; the ram weighed 900 pounds, and the gun 1,200 pounds—545 piles were driven in 17 days, at a cost of 26 cents each for cartridges.

2. For foundation walls of a government building at League Island Naval Station, with the same driver, piles 12 inches diameter at the butt,

* Communicated April 13, 1873.

25 feet long, driven their whole length into hard gravel bottom ; 966 piles were so driven in 22 working days, the greatest number in one day having been 81. The average quantity of cartridges used cost 32 cents per pile.

3. At Union Steam Forge of Macpherson, Willard & Co., near Bordentown, N. J., machine 60 feet high, cast-steel guides and afloat. The bottom there is acknowledged to be equal to the hardest within many miles of Philadelphia. In 4½ days, 141 piles were driven, of an average length of 30 feet, into a bottom that would have been impervious by the old process.

4. For landing wharf at League Island Naval Station. Here the first machine built was used, although its construction was so faulty as to lead to its abandonment. Notwithstanding the defects in the mechanical arrangement, that driver, in one day of 10 hours, in the month of March, drove 50 piles, averaging 9 inches bottom and 15 inches top diameter, 35 feet long, 21 feet into gravel, with 8 blows, using 9 ounces of cartridges. Some 800 piles were there driven with that machine, as could not have been by the steam driver.

5. For pier for Philadelphia & Reading R. R. Co., above Port Richmond, Philadelphia. In 14 days, 557 piles, averaging 10 inches middle diameter, 30 to 35 feet long, were driven from 12 to 16 feet into sand and gravel.

6. I have now* in charge the driving of about 300 piles for tramways leading to the exterior batteries at Fort Mifflin, near Philadelphia, for the United States Government. The material is hard sand, of a character always trying to machines.

In conclusion, I have no hesitation in expressing my unqualified satisfaction with the "Gunpowder pile-driver," as a contrivance for the economical, rapid and thoroughly successful driving of piles, without pointing or ringing, and always without injury in the way of crushing or splitting; hence their full bearing capacity is invariably secured and loss of timber certainly avoided. That the gunpowder process will, ere long, effectually supplant all its predecessors, seems to me inevitable, and as an Engineer, I take pleasure in working so admirable a machine in its infancy.

Mr. Charles Macdonald.†—It is to be regretted that the element of cost has not been considered more fully in the description of this method of pile driving by the use of gunpowder. The mere driving a pile after

* April, 1873.

† At the meeting December 17th, 1873.

it is placed in position in the leaders is frequently but a small item of the whole cost. It must be lifted from its bed alongside the driver, and conveniently suspended from the top of the leaders, while the lower extremity is placed in its proper position for driving; this can only be effected with economy, by the use of a steam-engine, which, with its attendance and fuel, is a source of constant expense; an important question is therefore, whether the time saved in driving piles by this method compensates for the extra cost of the powder, over and above that of the fuel which would have been consumed in performing the same work.

In the case at Greenwich Point, we find that an average of 32 piles per day were driven, at a cost for powder of $8.32. While at League Island an average of 44 piles per day were driven, at a cost for powder of $14.08. In the other cases mentioned, the cost is not given, but from this showing it will scarcely be asserted that the gunpowder pile-driver has developed any economic merit; judging from past experience, I have no doubt an average of 40 piles per day could have been driven, with a good steam machine, in either of the above cases, at a cost for fuel not to exceed that of half a ton of anthracite coal, which would represent a saving of about $7. I have never yet come in contact with circumstances under which such a marked difference in cost could have been compensated by the use of the gunpowder machine.

Prof. De Volson Wood.*—We have thus presented the extremes in the practical results of "gunpowder pile-driving." Were Mr. Probasco's paper only considered, great injury might result to a novel invention. It is evident that his machine was defective, or else it was improperly used, and the merits of a new invention should not be judged under such circumstances alone. A much fairer way is to compare the efficiency of a new machine, which works successfully, with the older ones. Mr. Holroyde was satisfied with the working of his machine, and yet the cost of driving piles with it appears to exceed that with the ordinary pile-driver. I believe, however, that it has merits which will make it especially serviceable in many cases.

* At the same meeting.

Fig. 1.

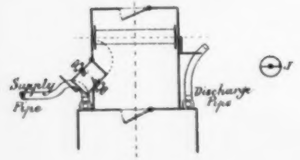
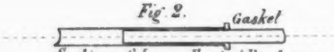
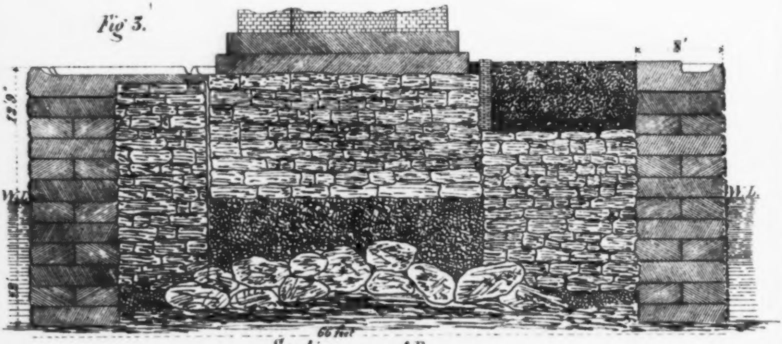


Fig. 2.



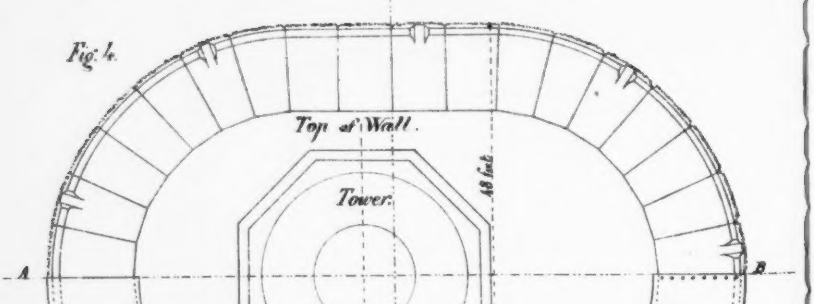
Section of Lower End of Discharge Pipe.

Fig. 3.



Section on AB.

Fig. 4.



LXXXI.

PNEUMATIC FOUNDATIONS.

A Paper by GEN. WILLIAM SOOY SMITH, Member of the Society.

READ FEBRUARY 19TH, 1873.

Up to 1859, but two bridges had been built in the United States on pneumatic pile foundations, one in 1855, over the Great Pedee river, where it is crossed by what is now known as the Wilmington, Columbia & Augusta R. R., and the other in 1857, over the Santee river where it is crossed by the North-Eastern R. R.

The air-lock used in sinking the piles constituting the piers of these two bridges, was the invention of Alexander Holstrom, and consisted of a cast-iron cylinder 6 feet in diameter (outside), 4 feet high and closed at top and bottom by cast-iron plates strengthened by flanges. Through these plates were circular openings (man-holes) 20 inches in diameter, closed by valves, both opening downward. A windlass was provided for raising the material to be excavated from within, all of which was removed by hand; 2 goose-neck pipes passed through the sides of this air-lock and down through the bottom. Through one of them, air was carried from the air-pumps into the cylinder, and through the other water was discharged, when the materials through which the cylinder was sinking was so impervious as to prevent the escape of the water under the bottom from the pressure of the air forced in. Light was admitted into the air-lock through 2 bulls-eyes of glass, 1 inch thick and 6 inches in diameter, and into the cylinder by 2 similar bulls-eyes, set in the bottom of the air-lock. The air pumps (of which there were 4 set in a single frame) were of such excellent construction, that after having been used for putting in the pier above-mentioned, they were again used in putting down the pneumatic pile piers of the Harlem bridge; and later still they did duty on the piers of the bridge over the Missouri river, at Leavenworth, Kansas. They are now being overhauled to be used in the construction of pneumatic pile piers for a bridge over the Arkansas river, at Little Rock.

In the fall of 1859, I began the construction of pneumatic pile piers for a bridge over the Savannah river, on the line of the Charleston &

Savannah R. R. The air-lock with which I commenced the work was upon the Holstrom plan; but it was 6 instead of 4 feet high, and the cylindrical portion was, for the sake of lightness, of boiler plate instead of cast-iron. Its diameter was the same as that of the cylinders to be sunk, viz., 6 feet (outside). I early discovered the following defects in my air-lock. 1st. The only natural light we could get into the cylinder had to pass through the bulls-eyes at the top of the air-lock, and then down through those in its bottom. As the latter were necessarily covered with dirt most of the time, the quantity of light thus admitted into the cylinder was so small as to be of no practical value. 2d. The storage capacity of the air-lock was so small that work in the cylinder was constantly interrupted, while materials were discharged from the chambers.

To obviate both of these defects I designed an air-lock with an inclined discharge-pipe or trap, through which materials were discharged as fast as they were brought up from the bottom. The construction of this lock is sketched in Fig. 1. A pipe was let through the side of the air-lock, at such an angle that all solid materials excavated would, by their own gravity, slide through it readily. It was closed on the inside by a hinged valve; the outer end was finished as shown at *J*, the lower semicircle of the base being hung upon a centre-pin, which was turned by a lever, when we wished to discharge the contents of the pipe. This threw the lower semicircle up behind the upper one, and left an opening through which the solid materials escaped. There was a small cock set outside, and one inside, as shown at *a* and *b*.

The operation of this subsidiary air-lock or trap was as follows: The outer end of the trap and outside cock being closed, and the inside open, the trap was filled. The inside valve and cock were then closed, and a signal struck on the inside of the air-lock; at this signal a man on the outside opened the outside cock, and the small quantity of compressed air contained in the trap escaped almost instantly. The lever was then given a half turn, and the contents of the trap escaped. Another half turn of the lever closed the trap on the outside; the outside cock was closed and a signal was struck from the outside. The cock through the inner valve was then opened—the trap filled with compressed air—the inner valve relieved of pressure was then opened, and the trap was ready for another charge.*

As we could get rid of the materials as fast as they were brought up,

*In my later air-locks I use 2 traps for discharging, and a small engine for operating the windlass.

no storage room was required, and hence the diameter of the air-lock could be reduced, so as to leave an offset at the bottom of the air-lock through which light was admitted by 6 bull's-eyes of the usual size. This enabled us during the day to dispense with candles and lamps, which burned dimly and with a suffocating smoke in the condensed air. At night, light could be reflected into the cylinders, through the bull's-eyes, and so the cylinders could be lighted day and night without the use of lamps or candles inside. Our work could be pushed without the interruptions that were so frequent while we used the Holstrom air-lock. The ratio of time required to do the same amount of work, with the old and new air-locks, proved to be as 14 to 5.

We had always used a pipe for discharging the water from the inside of our cylinders. One day I noticed that when this pipe reached the sand, where the water had all been discharged, a puff of sand followed. This was the falling apple. I at once procured a piece of flexible rubber hose, made upon skeleton rings, and fitted it to the bottom of the discharge-pipe; then, when we had discharged the water from the cylinder, we thrust the end of this rubber hose into the sand and allowed a current of compressed air to raise the sand through the pipe, and discharge it on the outside. This worked like a charm. The ratio, which before was as 14 to 5, became as 28 to 1. Instead of a piece of rubber hose at the bottom of the discharge-pipe I have used a telescopic section of iron pipe, shown in Fig. 2, with still better results; the friction on the inside of an iron pipe being less than that in a rubber one; and there is an advantage in reducing the discharge-pipe at the lower end, which is done by slipping the telescopic section inside of the discharge-pipe. I put a plaited hemp gasket between them, by making the gasket so much larger than the opening between the pipes that it cannot blow through. With this arrangement, and a gang of but 7 men, I have excavated 6 cubic yards per hour, for several hours in succession. Two-thirds of a cubic yard per day, per man, is about the rate of excavation by hand, or without a sand-pipe.

The war interrupted the work on the Savannah river bridge before completion. It also prevented further consideration of a plan for the construction of a light-house on Frying Pan shoals, or a similar situation on our coast, which, in 1860, I brought to the notice of the U. S. Light-house Board. This plan embraced the construction of a caisson, of large size—say from 30 to 50 feet in diameter—which I proposed to sink by my improved method to any required depth less than 100 feet.

Inside of this, I proposed to build a foundation of masonry, consisting of large stones joggled or dovetailed, like those laid in the Eddy Stone Light-house. Soon after the close of the war, I again submitted this plan to the Light-house Board, General O. M. Poe being the Engineer Secretary, and I was requested to adapt it to the repairing of the Waugoshance Light-house, for which an appropriation had been made.

This light stands at the western entrance to the straits of Mackinac, upon a rocky reef $2\frac{1}{2}$ miles from shore. The tower is of brick, 20 feet in diameter, and 84 feet in height from the water surface to focal plane. It stands upon a foundation 24 feet square, which consists of a crib filled with concrete and rubble masonry. This crib is surrounded by others, which, with it, were all framed into one pier 100 feet square, and fitted to the bottom, according to accurate soundings previously taken. The surrounding cribs were filled with loose stone. During heavy southwestern gales, the pier was exposed to the action of as heavy seas as are to be found anywhere upon the northern lakes—the spray frequently going entirely over the tower.

The work was first constructed in 1848. The timber above water had partially decayed, and the heavy seas, and still more hurtful ice action, were injuring the work seriously, and threatening its complete destruction. Under these circumstances, it became necessary to throw a protecting wall around the tower, of sufficient strength to resist the action of seas and ice. To do this I designed an iron pneumatic coffer-dam of the form sketched in Fig. 4, which was built up in place, in a chamber cut down to the water surface through the old crib. The sizé of the old crib work was such, that a sufficient temporary protection was afforded to the work, while in progress, by the portion of the piece left intact outside of the chamber cut, to receive the dam. Two air-locks (one shown), with rectangular traps set through their ends, were provided ; these traps had valves and cocks inside and out, the inside valve opening outward from the trap, and outside valve opening inward. Through these traps all the materials excavated, and the hands employed, were readily passed. The materials were raised by windlasses, operated by small hoisting engines, bolted to the outside of the air-locks.

Heavy beams were placed across the deck of the dam, resting upon blocking between the dam and tower and outside the dam on the crib-work. From these beams the dam was suspended by chains attached to hooks, taking hold under the bottom edges of the dam inside and outside. The dam was then heavily loaded with stone, the load being in-

creased as it descended, so that the weight of the dam and its load was constantly kept slightly in excess of the weight of water displaced by the dam and its compressed air filling, the excess being held by the suspension chains.

When all these preparations had been made, the work of sinking commenced. Throughout the first 6 feet below the water surface, the timber walls of the cribs had to be cut away and removed. These walls consisted of 12 x 12 inch pine timber, built up solid and fastened very strongly with round drift-bolts of an excellent quality of iron, one inch in diameter. These were rusted in, and the heads were frequently pulled off by the powerful claw-bars used, before they could be started, in which case they had to be cut or sawed out. The work of cutting through this crib work was very tedious, and about 6 weeks were consumed in accomplishing it.

We then came upon the bed of boulders forming the reef. These were of various sizes, from that of a hen's egg to 10 tons weight, and some of the largest lay under the edges of the dam, in all sorts of shapes and positions. Whenever we could readily get at them, we split them with plugs and feathers. In other cases we undermined and drew them into the dam, and then split them. After going down into this boulder formation about 3 feet, some large stones rolled in against the dam in wedging positions, preventing it from sinking further. We then partly unloaded the dam, and permitted it to rise. The troublesome stones rolled in and were split up, and thus gotten rid of. This occurred twice afterward, and the difficulty was treated and overcome as in the first instance.

We finally succeeded in sinking the dam to a depth of 12¼ feet below the water surface and 6 feet below the foundation of the tower; at this depth we expected to find bed-rock, as it occurred along the contour of the reef of boulders, at that depth in a line nearly surrounding the Light, and at a distance of from 200 to 500 feet from it. But there seemed to be a depression in the surface of the bed-rock just where the Light was situated, and we were disappointed in reaching rock in place at the depth we expected.

The boulders grew less and less in size as we descended,* and at the depth reached they were adjusted to each other most perfectly, after the manner of a cobble-stone pavement. As the portion of the crib work

* Can any member account for this.

outside the dam was to remain, and as it will never decay under water, it constitutes an excellent protection from any undermining action of the waves, even if we could imagine that any disturbance of the reef would ever take place at a depth 6 feet below its general level. No such digging action of the sea had been observed to undermine the old cribs that had stood there for 20 years.

It was therefore determined to go no deeper. The work of sealing the coffer-dam at the bottom was next commenced. Louisville cement of the quickest setting kind—dry—was spread evenly on the bottom, to a depth of 6 inches. The air pressure was then allowed to run down until the water entered and covered the cement; several holes, say 2 feet in diameter each, being left to permit the expulsion of the water after this layer of pure cement had set. When this had taken place—under the water and not in the compressed air—the water was forced out by air pressure, and another layer of cement 6 inches thick was added and treated as before. I believed that this floor one foot in thickness would be sufficient to resist the pressure of the water 12¼ feet in depth on the outside; after the water had covered the surface 4 days, it was again forced out until even with the top surface of the cement floor. The holes previously left were then closed with cement and it allowed to set.

When the bottom was thus completely sealed, the air pressure within the dam was allowed to escape slowly; long before it had ceased to act in aid of the bottom, the latter failed. I regret that I did not note the reading of the pressure gauge at the moment of failure, from which the strength of the cement bottom could have been accurately calculated. As the stones were contracted for and partly cut for a given height of wall, and as it was desirable to carry the masonry itself to as great a depth as possible, I determined not to add further to the thickness of the cement bottom. One of the cap plates of the dam was removed, and converted into a valve, through which the stones for the sea-wall were lowered, 3 or 4 in succession, when the valve was closed and these stones laid. After 3 courses, 2 feet thick each, were laid in this way, it was found that the bottom was sealed so that the water no longer entered. The cap sheets were then removed, and the remainder of the wall was laid up in the open air. The coffer dam, which might have been removed for use elsewhere, was left in place, to give additional strength to the wall.

Each stone was secured in place by an iron dowell, 2¼ inches in diameter. A hole 1 inch in diameter and 3 inches deep was drilled in each

end of the dowell, which was cross-sawed. When in place the dowell was expanded by a tapering pin driven firmly into the hole, and thus the stones were fastened together so that they could not be removed without splitting them or breaking the dowell. When the wall was built up in this way, the space between it and the tower was filled with concrete, and covered with the flagging and coping course, as shown in Figures 3 and 4.

Work could only be carried on during the 6 months—from May 1st to November 1st—and in this period it was frequently interrupted by the seas, which broke over the site during the prevalence of storms. The force employed averaged 40 men. During the first season the chamber to receive the dam was excavated, the machinery put in place, and the dam built and sunk 4 feet. During the second season, the dam was sunk to the required depth and 7 courses of stone laid up, and the work was entirely finished the third season. The total cost, including a new dwelling for the light-keeper, was less than $200,000. I have gone thus into details, as it is the first instance, in this country, of the sinking of a pneumatic coffer-dam or caisson, and was otherwise characterized by many new features.*

From the Waugoshance Light-house I went, in 1869, to Omaha, to sink the pneumatic pile piers for a bridge over the Missouri river at that point. These were the first pneumatic piles ever sunk in this river, and, indeed, the first west of the Alleghany mountains. They were to be put down to a greater depth than, up to that time, had ever been reached by this process anywhere, namely, 82 feet below the water surface, mostly through finely comminuted silt, interstratified with thin deposits of coarse sharp sand, and layers of tough blue clay; the latter not exceeding 2 feet in thickness. Next to the bed-rock there was a stratum of gravel, consisting of well-rounded pebbles, from 1½ to 2 feet in thickness. These materials presented the most difficult features met with in sinking pneumatic piles.

The tendency to lurching out of a vertical position was such that I found it impossible to prevent it in all cases. The first cylinder sunk went down to the bed-rock in a vertical position, all right; the second sank vertically 27 feet, then took an inclination which we were unable to correct by all known means we could employ while sinking it

* I presume also that members of the Society will generally agree with me that the best part of the description of works is that which gives details.

through the next 20 feet. It may be of interest to the profession to learn the nature of the methods adopted to correct this inclination ; for although they failed in this particular instance, I have, in many other cases, succeeded with them perfectly, even after a pile had sunk to a depth of from 40 to 50 feet. We first excavated the sand to the bottom of the cylinder, placed heavy wooden wedges under its edges on the deepest side, and then suddenly relieved it from upward pressure by permitting the compressed air to escape. I rely upon this method of straightening a cylinder with more confidence than upon any other ; but in this instance it failed. The influx of silt was so great that the wedges were brought in with it, and so offered no extra resistance to the sinking on the lower side. We then added a section of our cylinder, 10 feet in length, thus making the pile 16 feet above the surface of the surrounding earth. A strong frame, of 12-inch square yellow pine timbers, was then placed around the cylinder, at the surface, and carried out to such a distance that it could be secured to the earth at points beyond any disturbance from sinking around the pile ; this afforded a fulcrum. With blocks and falls attached to the top of the air-lock, which was 22 feet above the surface (the air-lock being 6 feet long), we put as heavy a pull as we could upon the pile, without danger of breaking. The sand was then excavated to the bottom ; a beam, in the form of a segment of a circle, was then inserted under the lower edge of the cylinder, the air was allowed to escape, and the pile descended. The timbers of the fulcrum, 9 feet between bearings, were broken, but the inclination was unchanged. A pine strut, 8 inches square and 11 feet long, was then set at a slight angle, with its foot abutting against the cylinder already sunk, and toward which the one in hand leaned. With this we also failed. The cylinder finally broke off 27 feet below the surface, where there was half way around it a "cold shut" in the metal.

The upper piece of the broken cylinder we had then to lift out. When we commenced, there was 15 feet of sand on the inside. I thought we could raise it easily with the air-pressure we could force in ; but, up to a pressure of 45 pounds to the square inch, the cylinder was not started. I feared to use more, lest the bolts that secured the air-lock to the cylinder should break. The sand was then excavated from within, and the cylinder lifted with a pressure of 27 pounds to the square inch. This gave me the best measurement of the friction, on an iron cylinder sunk into the earth, that I have ever been able to obtain.

The inside area of the air-lock was 7,700 square inches, its weight 14,000 pounds, and that of the cylinder 48,600 pounds, whence the upward pressure, in pounds, was

$$(7{,}700 \times 45). - (14{,}000 + 48{,}600) = 346{,}500 - 62{,}600 = 283{,}900.$$

Before the sand was removed from the interior, there were 160,000 square inches exposed to earth pressure, whence the friction, in pounds, per square inch, was greater than $\frac{283{,}900}{160{,}000} = 1.77$. How much more than this it was could not be ascertained, as the friction, in this instance, was not overcome. After excavating from the interior, the surface exposed to earth pressure was 103,824 square inches, and the pile was removed by an upward pressure, in pounds, of

$$(7{,}700 \times 27) - 62{,}600 = 145{,}300,$$

whence the friction, in pounds, per square inch, was about $\frac{145{,}300}{103{,}824} = 1.39$

The friction per square inch, overcome, when the pile was removed, was less than it would have been before the excavation was done, as during the removal of the 15 feet of sand from within the cylinder, the materials outside were disturbed.

Of course the friction per square inch will increase with the depth in a semi-fluid mass, such as the cylinder in question was sinking through. The law of such increase, however, is not definitely known. The foregoing determination of the friction, derived from a single measurement, obtained under peculiar circumstances, and such as do not exist when a pile is called upon to act as a supporting column, with nothing but friction to sustain it, can only be taken as a guide in estimating what that friction really is. A very important subject of inquiry is what resistance friction alone makes to the sinking of an iron cylinder of a given size, in sand, to a given depth. Where no bed-rock can be reached, as at points on the lower Mississippi river, and in the neighborhood of the Gulf coast, it may become necessary to build piers consisting of piles or caissons resting wholly in and on sand. In some instances piles have been sunk by hydraulic pressure, so applied as to force them down without allowing the compressed air to escape. The resistance overcome in these cases would give some idea, though not an accurate one perhaps, of the friction developed.

We put down the next 2 cylinders without any extraordinary difficulty, one of them at the rate of 10 feet per day. To cause the cylinders

to sink, it was necessary to load them ; this we did by building frames, upon the inside flanges, at each joint, and filling the cylinder with stones, except at the centre, where a well, 3 feet square, was left, which was lined with 2-inch plank, spiked to frames made of 2 x 4-inch scantling. Through it the men passed to the bottom, and the materials excavated from within were raised by the air-lift already described. It was found that the cylinders could not be loaded with stone sufficient to make them follow the excavation as it proceeded ; therefore to make them descend a portion of the compressed air was allowed to escape suddenly. The lower section of each cylinder was closed at the top with a cast-iron diaphragm, made in 4 flanged sections, and bolted together ; through it there was a man-hole, 20 inches in diameter, closed by a valve opening upward. When the sand was excavated to the bottom of the cylinder, the workmen came up from the lower section, the valve was closed tight, a portion of the compressed air was allowed to escape, and the pile sank from 2 to 4 feet each time ; then the pressure was restored, the valve opened, and the work of excavating resumed. Thus no more sand could at any time enter at the bottom than would fill the lower section. The outside materials were less disturbed than before, and the cylinder more readily kept in a vertical position.

At this stage of the work I left it, to commence the Leavenworth bridge, which was to be built upon a similar plan. Mr. Theophilus E. Sickles took charge of and completed the Omaha bridge ; I have learned that he employed levers for forcing the cylinders down, and that he succeeded well in rectifying any inclination which they took, by drilling through them at different points on the higher side, and thus allowing currents of compressed air to escape, which by disturbing the outside materials decreased the pressure on that side, so that the cylinders righted.

The operations at Leavenworth were characterized by difficulties similar to those experienced at Omaha, though here we had to go to a less depth, and had our former experience to aid us.

The Leavenworth bridge was completed within 2 years from the time of its commencement. It consists of 3 spans, each 340 feet long, resting upon a stone abutment on the west bank, and 3 pneumatic pile piers—2 in the river and 1 on the east bank—the approach to the bridge at this end being by a trestle.

The following conclusions result from my study and observation on

this subject during an experience of 14 years in sinking pneumatic foundations:

First.—The greatest difficulty encountered in sinking pneumatic piles is to overcome their tendency to take an inclined position while sinking, and to correct this inclination after it occurs.

Second.—The best way to resist this tendency is to force the pile down without letting off the pressure, so that it will follow the excavation at the bottom.

Third.—Wedging under the bottom of the pile and lifting its top on the lower inclined side, to which may be added the air-jets used by Mr. Sickles, are to be regarded as the most effectual means yet tried for correcting inclinations.

Fourth.—The air-lift is by far the cheapest, most rapid and best method of excavating sand or mud, from within a pneumatic pile or caisson.

Fifth.—A very strong and perfectly reliable pier can be built of pneumatic piles of proper diameter and thickness of metal; to be determined, as well as their number and relative positions, by the circumstances of the case.

Sixth.—The permanence of a pneumatic pile pier may be greatly endangered by liability to fracture by frost. This, of course, is to be apprehended only in cold climates; I recommend as a preventive, the use of a filling below the frost line, of asphaltic concrete, from 2 to 5 feet deep.

Seventh.—Where dimension stone is abundant, and timber available at a reasonable cost, a pneumatic caisson, surmounted by masonry, is cheaper and better than a pneumatic pile pier. A single caisson can be sunk to a given depth quicker than the 3 piles which usually make up such a pier, and with less difficulty and uncertainty as to time than have attended the sinking of these piers the world over.

Eighth.—The cheapest and best bridge pier yet devised, where it has to be sunk through considerable depth of soft material to a harder one, is the cellular wooden caisson, the outside and cross walls being well drift-bolted, with working chamber at the bottom, the whole sunk by the pneumatic process, filled with concrete or rubble masonry up to within 5 feet of low-water surface and finished to bridge seat with masonry.

Ninth.—In my experience, concrete will not set satisfactorily in compressed air, therefore I have allowed the water to enter and cover it be-

fore setting. This is best done through a pipe, reaching down to a level with the bottom of the cylinder, which prevents the passage of water through the mass of concrete. After filling a cylinder in this way to a depth of 5 feet, I have sealed it, then set shores from the top of the concrete to the first flange above, let off the pressure and completed the filling in open air.

It was not thought desirable to give in this paper the oft-repeated description of the method of sinking pneumatic foundations, and of the pumps and other machinery employed, nor to specify dimensions of cylinders and details of their construction. If I have given any facts which will aid a member in planning a foundation, or in putting it in, my purpose is accomplished.*

Mr. C. C. Martin.—Gen. Smith mentions the great difficulty in keeping pneumatic cylinders vertical while sinking, and the greater one in restoring them to the vertical when inclined. It was my fortune, in 1861, to follow him in charge of sinking the cylinders for the bridge piers on the Savannah river, on the line of the Charleston & Savannah R. R. The first work I had to do was to straighten up a cylinder which was about 30 feet in the sand at the bottom of the river and considerably inclined. I tried several times at first, the method advised by him, and which he had previously used with some success, to excavate the material inside of the cylinder to the bottom, drive wooden wedges under the lowest edge of the cylinder, to prevent that side from going further, and then allow the air quickly to escape, the cylinder to move down, and in the operation to tend towards the vertical, which tendency was increased by a powerful tackle. After several trials, during which the cylinder moved downward about 2 feet, it became evident that this method would not succeed. I thereupon decided to try the following plan. I excavated the material to the bottom of the cylinder and drove the wedges as before; then excavated underneath and outside of the cylinder, on the side opposite the wedges, as far as was possible without causing the sand to fall down from the outside. This caused all the air that escaped under the bottom of the cylinder to pass out on the side opposite the wedges, and thus loosen the material against which the cylinder would have to move, in being righted. This, it will be observed, accomplished what Mr. Sickles effected at Omaha by having holes

* Should any member desire drawings of machines, to be used in sinking pneumatic foundations, of the piers themselves, or any other information that I possess, I will freely send what I have.

drilled in the upper side of the inclined cylinder. I then arranged a tackle to pull the cylinder towards the vertical, and also a battering ram, with which blows could be struck against its top to assist in righting it. This ram consisted of a stick of oak about 12 feet long and 12 inches square, suspended to the sheer poles above by a rope near the middle. When these preparations were all made and the men properly stationed, the air was allowed to escape; while the cylinder moved down as usual, the tackle was made taut, but though the wedges did their work, the cylinder did not change position. The battering ram was then set in motion, and at the second blow the cylinder began moving towards the vertical—the continued strain of the tackle and the blows of the ram soon brought it into position without further trouble. The jar or shock given to the cylinder seemed to be all that was required in addition to the strain on the tackle. I had no opportunity to test the operation on another cylinder, for the next one went down without leaving the vertical, and before work on a third was commenced operations were permanently suspended.

My impression is, that in all cases where practicable, it is much better to provide a platform from which to sink cylinders than to work from scows, especially in tide-water. In rivers liable to sudden and great changes in elevation of water surface, this plan would not be safe, as the whole plant would be liable to submergence and destruction.

Gen. W. Sooy Smith.—In view of the injurious effects of compressed air upon laborers, who excavate in the working chambers of pneumatic piles and caissons, after such are sunk to a depth of about 60 feet, I have given much thought to a plan for sinking these foundations through *sand*, without the necessity of sending men inside the compressed air chamber at all, except to remove a boulder, log or other unusual obstruction.

In the case of a pneumatic pile, there should be a short section at the bottom, provided with a cap containing a man-hole closed by a suitable valve, a supply-pipe and one or more discharge-pipes. The latter should pass through the cap by a universal joint arrangement, similar to that in Ward's water-pipe, and each having a telescopic section at the bottom. This would permit a change in the direction of the discharge-pipe, so as to make its foot reach any point on the bottom of the pile when the pipe is extended by means of the telescopic section. To permit freedom of motion above the cap, a flexible section should be inserted in the discharge-pipe.

With this arrangement, air could be forced into the short bottom section through the supply-pipe, and the excavation done by my air-lift, through the discharge-pipes provided for that purpose as already described. There would then be no necessity to send a man inside, unless to remove some unusual obstruction. The laborers required to manage the discharge-pipes could stand in the open air, upon the cap of the bottom section. The cylinders could be partly filled with masonry to increase their weight and aid the sinking.

Of course, this method could not be employed where the materials through which the pile is to be sunk are of such character as not to yield and flow with an air-current. In a caisson, the method would be the same except that more pipes would be required. The nature of the materials, and the presence or absence of logs, boulders or other obstructions can be ascertained by preliminary borings. Some such method as this may become necessary in sinking foundations to very great depths. The greater the depth, the more efficient the air-lift becomes.

Mr. Francis Collingwood.*—Gen. Smith states that in his experience concrete sets badly under pressure; this was not the case in the caissons of the East river bridge, where, on the New York side, the pressure was 35 pounds, and on the Brooklyn side 15 pounds per square inch; in each the concrete set well and with no less facility than in the open air. In filling the Brooklyn caisson, the first attempt was to prevent water from coming in, by making a dry floor about 25 feet wide from the cutting edge inward; the concrete for this was put down in layers of 6 inches thick, for a depth of 12 to 18 inches. This was ineffectual, the water would follow the timbers, rise to the center of the caisson, and stand in the lowermost places, from whence it was blown out through a flexible hose. Over this floor, the filling was continued in layers from 6 to 10 inches thick. In the Brooklyn caisson it was made of 1 part cement, 2 parts sand and 3 of gravel, and in the New York caisson 1 more part of gravel was added.

When concrete is laid under pressure, air will pass through it outward, or water through it inward; probably the first hastens the action of setting, and the second, by separating the cement and sand, retards or prevents this action.

After the fire occurred in the Brooklyn caisson it was necessary to remove several yards of the concrete; although none of this had been laid

* At the meeting, December 17th, 1873.

longer than four weeks, the portion taken out was quite hard. The workmen tried first to extinguish the fire by stopping the crevices with concrete; when, afterwards, parts of the caisson roof were cut out to make the necessary repairs, the concrete exposed was very hard and strong, and in some places quite difficult to detach from the timbers; wherever examined it had set uniformly throughout the whole mass.

HON. WILLIAM J. McALPINE.*—When I began filling the pneumatic piles of the Harlem bridge, the concrete was thrown in, 5 to 6 feet in depth at a time—which in 24 hours was penetrated by the water as thoroughly as sand or earth would have been; this was under a head of 40 feet, and the pressure exceeded that due to the water-head, by whatever was due to the friction of air escaping through the surrounding soil. Upon examination it was found that a crust (broken in some places) had formed on top of the concrete, which prevented the air from permeating the mass, and thus excluding the water. On reference to authorities, I learned that when the Saltash and Rochester (England) bridge foundations were put down, the concrete used did *not* set under pressure.

To determine what was the difference in action, when the conditions (except the pressures of the surrounding atmosphere) were alike, I sent samples of concrete into the cylinder, and found that setting took place in about one-half the time it did in the open air.† Afterwards, the concrete used was lowered through the air-lock in canvas bags, holding about 1½ cubic feet; it then set so rapidly that the bags could not be emptied without cutting.

To overcome the infiltration of water, and permit the concrete to harden under pressure, pieces of frost split gas-pipe were placed vertically in the cylinder, and the concrete filled about them; thereby the pressure was transferred to the bottom of the cylinder and the requisite quantity of air furnished for setting and crystallization. That this was effective was seen in one instance, where the concrete was cut into for 6 feet in depth, and found to be uniformly solid throughout.

From the effects of frost and moisture some of the cylinders of this bridge are cracked lengthwise, and also around under the flanges.

When building the United States Dry Dock at Brooklyn, it was necessary to exclude water which flowed in from springs, say 40 feet above the work, or under about 80 feet head. Concretes, pure and mixed, made with

* At the same meeting.

† As recorded in "Journal of Harlem Bridge," in the McAlpine Library of the Society.

the best varieties of American cement, were put down in succession and kept in place by a plank floor over the layers; these all failed. Perkin's Roman cement—an English artificial cement—was then tried with success. The inverted arches of this work, made with stone so carefully dressed that the joints were less than $\frac{1}{8}$-inch thick, gave similar trouble. These were made tight by driving in with a caulking iron a mixture of cement and sand, intimately ground together in a mortar and used dry.

Mr. Octave Chanute.*—In putting down the piers of the Illinois and St. Louis bridge, the air pressure was considerably greater than that due to the head of the surrounding water; no trouble was experienced with cement setting under pressures varying from 44 to 51 pounds.

In preparing such foundations, concrete in large masses should be allowed sufficient time to harden properly; French engineers say that heavy weights should not be put on concrete foundations in less than one year.

The piers at Omaha and Leavenworth are also cracked by frost; to prevent this, the piles might be lagged with wood on the inside between the flanges, so as to leave an interior cylinder of uniform section to be filled with concrete.

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* At the same meeting.